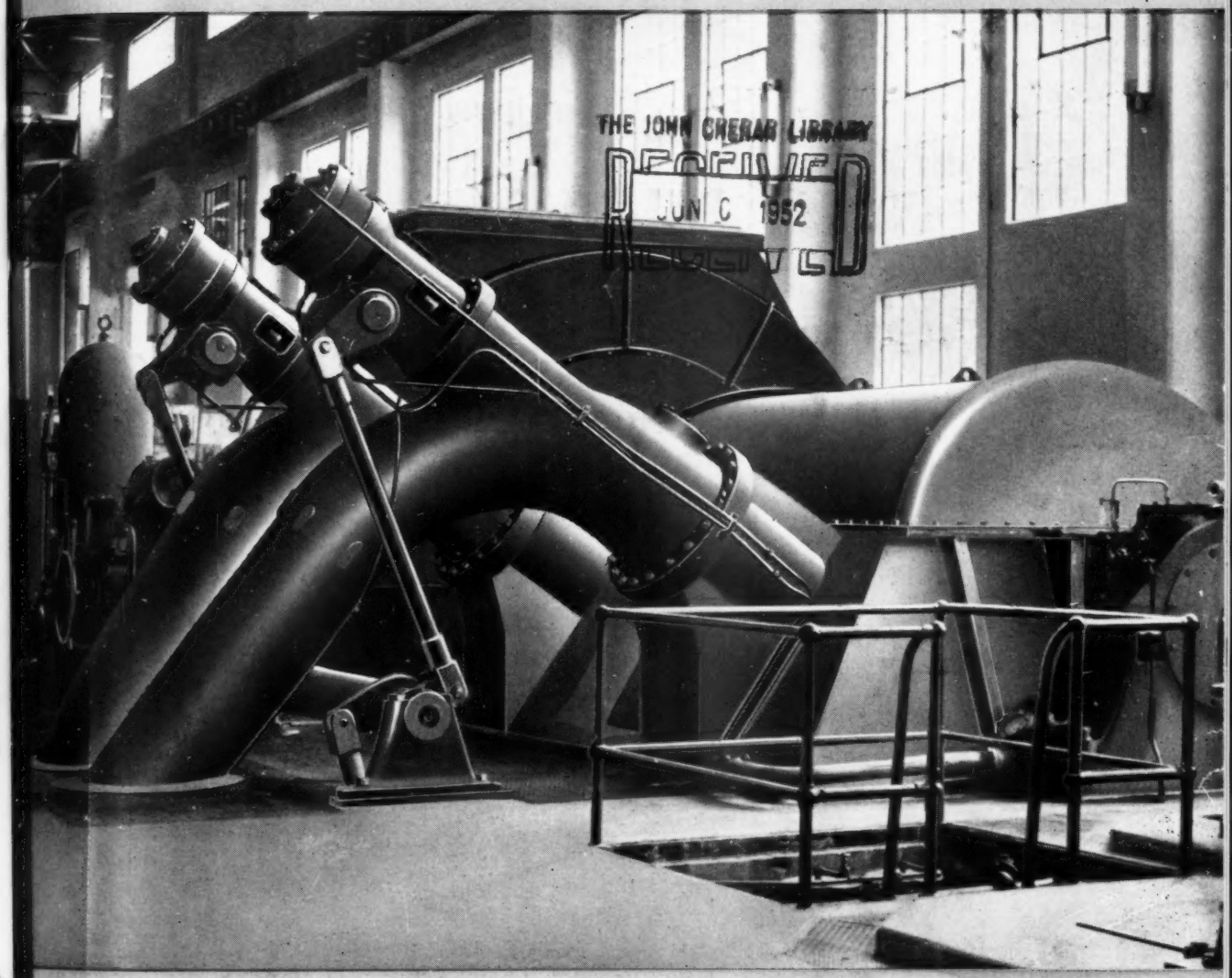
627

Water Power

INTERNATIONAL DEVELOPMENT OF HYDRO-ELECTRIC POWER

PRICE 2/6 : JUNE 1952



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A 21,000 h.p. double runner twin jet impulse water turbine driving a 16,750 kVA 11,000 volts alternator in Tasmania.

NGLISH ELECTRIC'

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The ENGLISH ELECTRIC Company manufactures complete hydro-electric generating plant ranging from main turbines and alternators to the smallest auxiliaries, including the transformers and switchgear. 'ENGLISH ELECTRIC' plant has been installed in all parts of the world under widely diverse conditions, meeting perfectly an extensive range of requirements.

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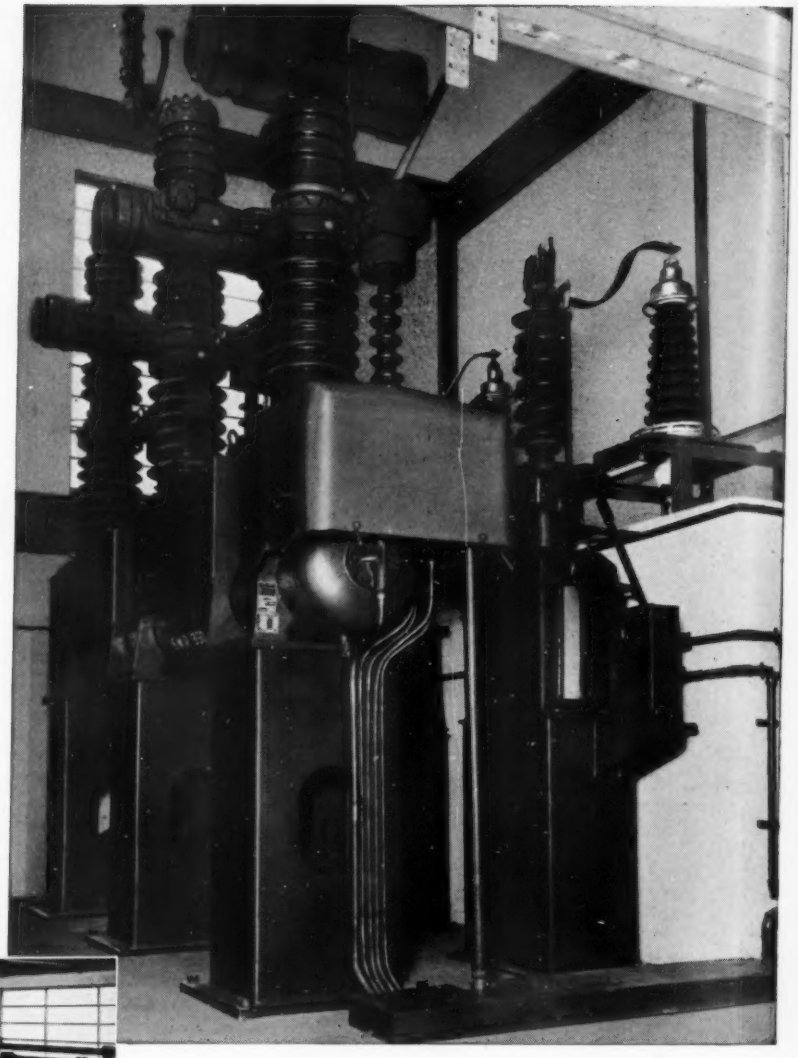
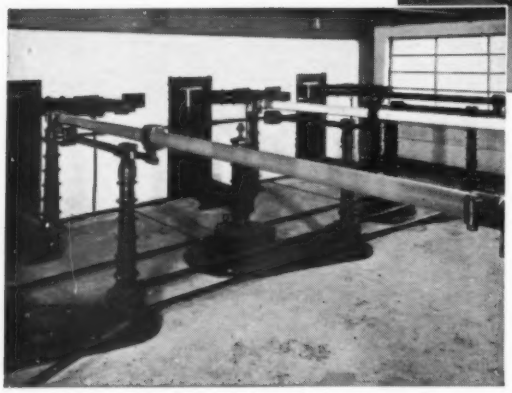
★ *If you want to talk switchgear . . . . talk to Reyrolle*



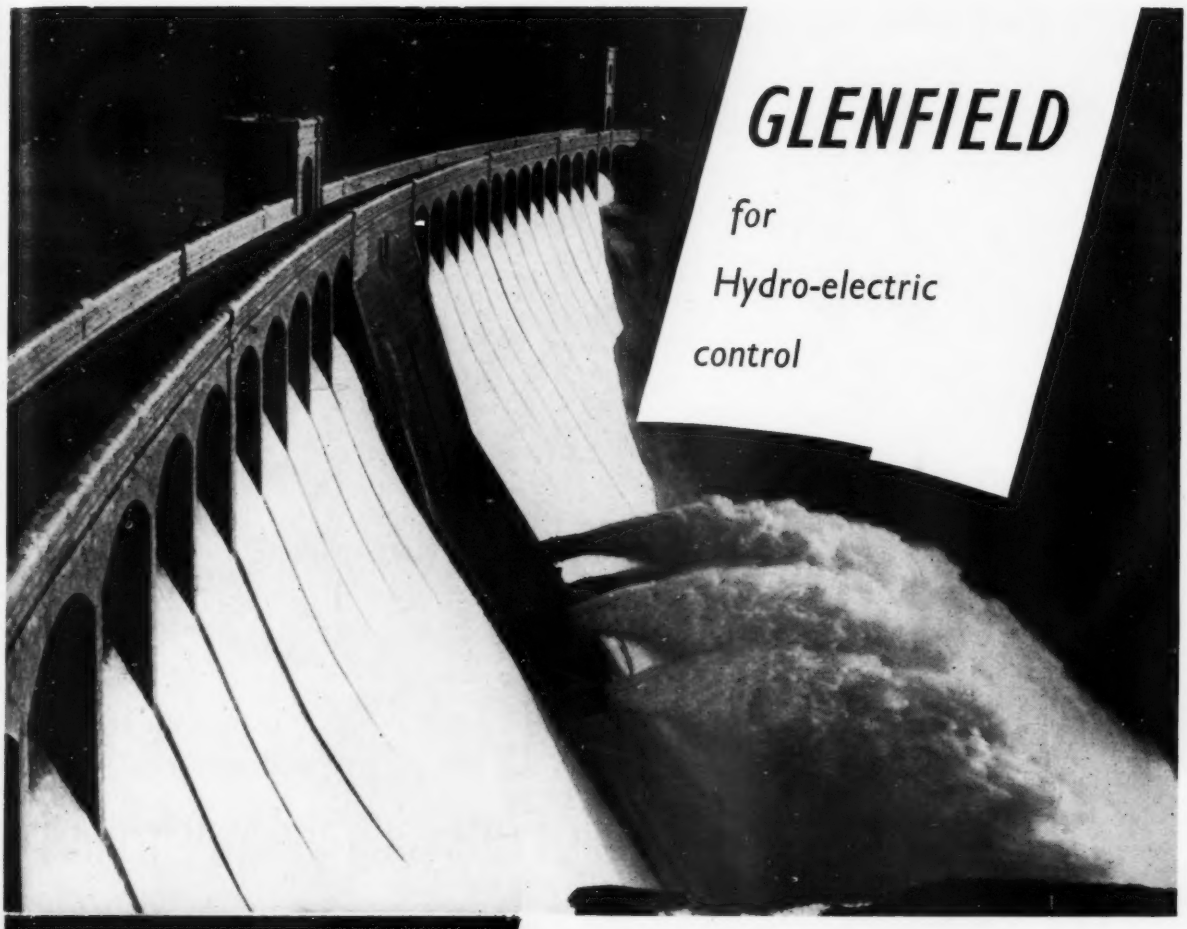
AIR-BLAST SWITCHGEAR AT NORTH TEES



The indoor installation of 66 kV air-blast switchgear at the North Tees Power Station of The British Electricity Authority (North Eastern Division) has circuit-breakers of 2500 MVA breaking capacity. There is a ring-busbar system in chambers above the circuit-breaker compartments. Each three-phase circuit-breaker, standing on a pedestal to give the necessary clearance, is mounted in a separate compartment, bushing-type isolators which are earthed in the open position are mounted in the compartment walls and allow safe access to equipment which has been isolated.

If you want to talk switchgear ··· talk to Reyrolle

A. Reyrolle & Company Ltd., Hebburn, Co. Durham.



GLENFIELD

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Hydro-electric

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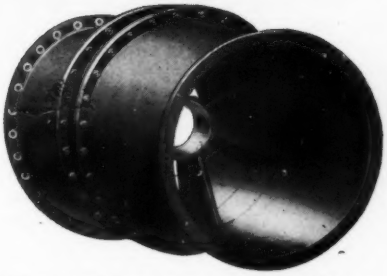
JET DISPERSERS



GLENFIELD JET DISPERSERS are in use throughout the world on points of free discharge.

On siphon spillways, scours and on compensation water outlets from dams and reservoirs, and washouts from pipelines.

To result in the harmless dissipation of the issuing jet, without vibration or erosion either of the disperser itself or of the area on which the discharge falls whether into river bed—tunnel—or ditch.

To render elaborate and costly protective works unnecessary.

The top illustration shows six jet dispersers on a siphon spillway each designed to discharge at the rate of 620 cubic feet of water per second to absorb in turn a total of 25,000 H.P. in the process.

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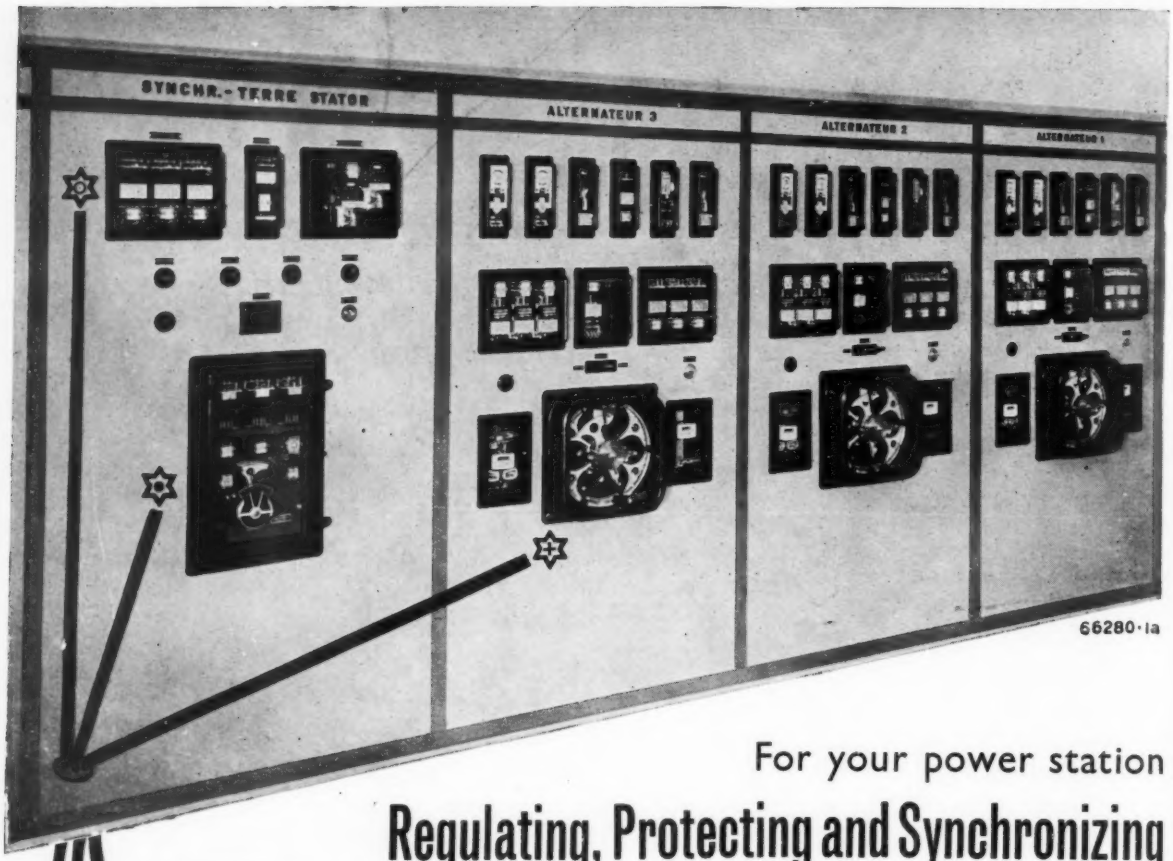

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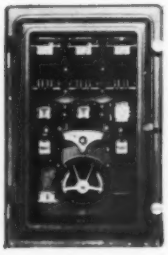
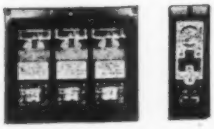
Quick-acting regulators
A suitable model for every size of machine and every regulating duty. Accurate and robust.



Automatic synchronizers
for generating stations and substations; require little space, no maintenance, no adjustment; low power consumption. Rapid and reliable.



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Machines with an aggregate rating of millions of kVA are synchronized and protected by Brown Boveri equipment.

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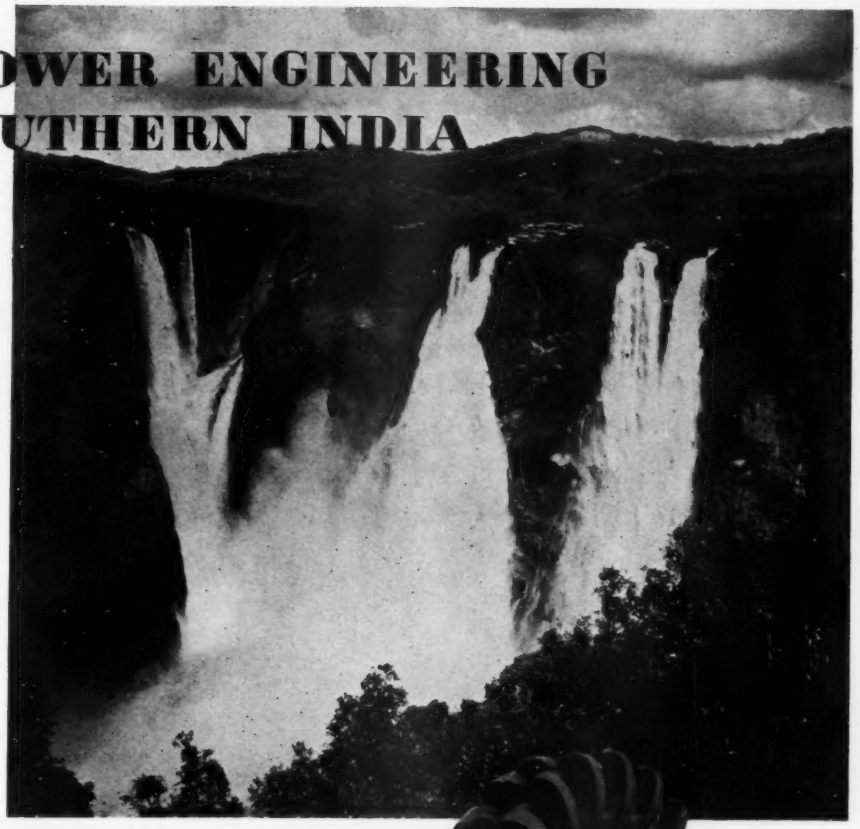
WATER POWER ENGINEERING *in* SOUTHERN INDIA



The first four 19,000 h.p. Pelton Turbines installed for the Government of Mysore at the Jog Falls Station came into commercial service at the end of 1948.
Further extensions to this Station comprise four double runner 4-jet horizontal shaft Peltons, each designed to develop 32,500 h.p. under a nett head of 1190 feet.

Manufactured at the Works of Markham & Co. Ltd., Chesterfield.

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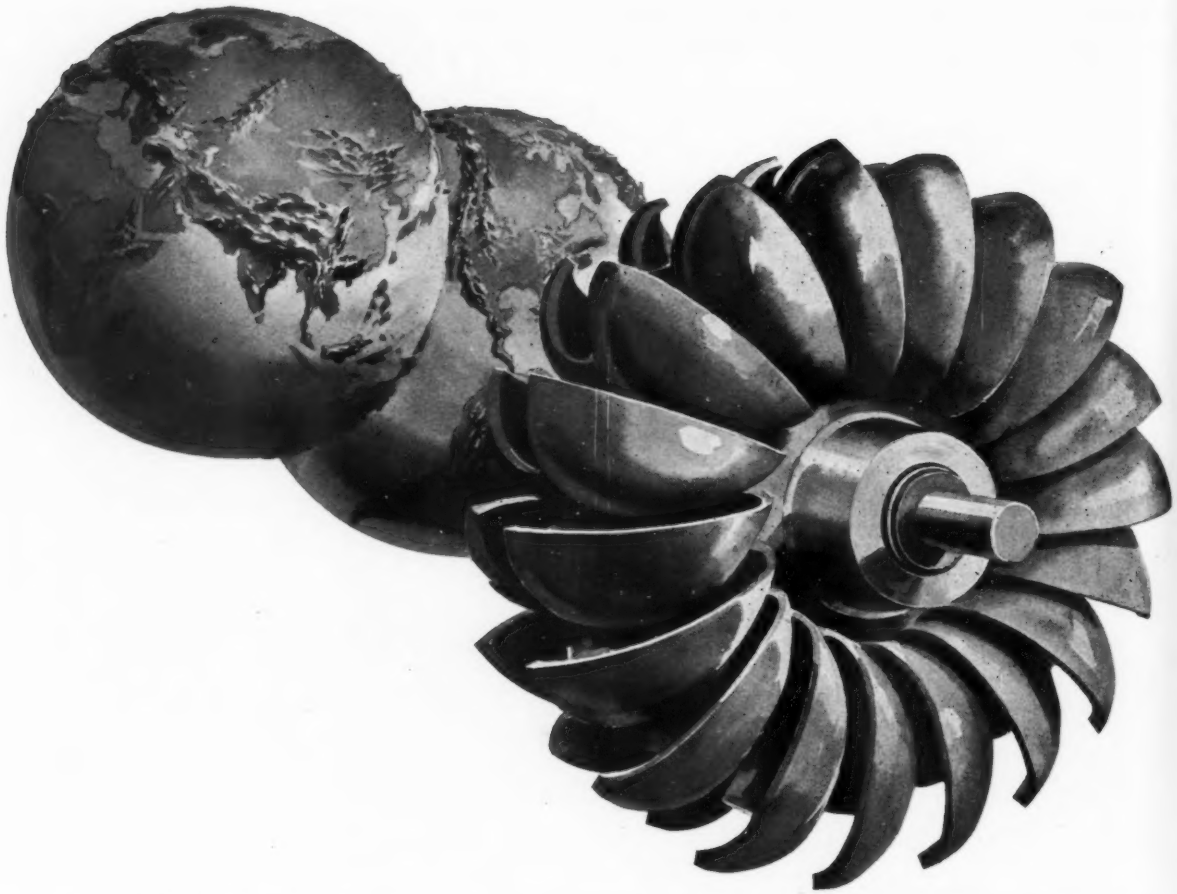

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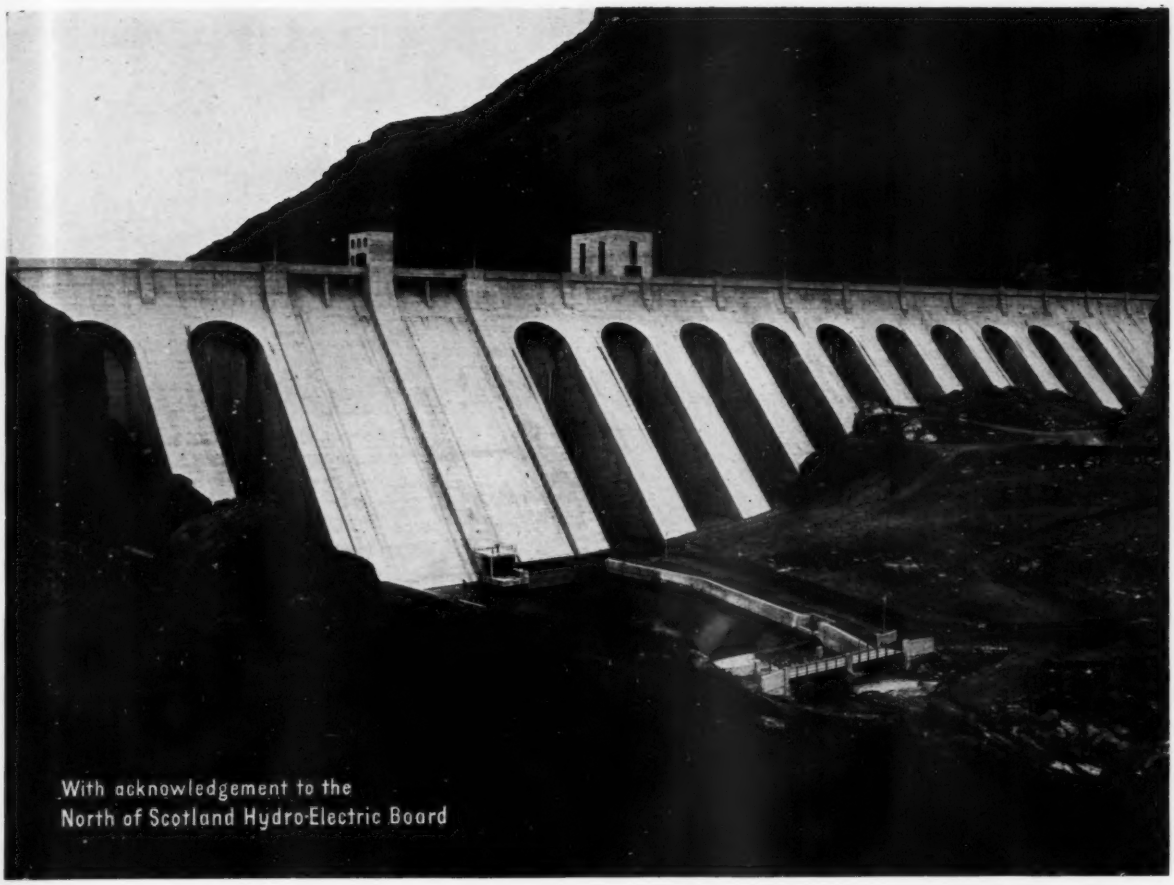

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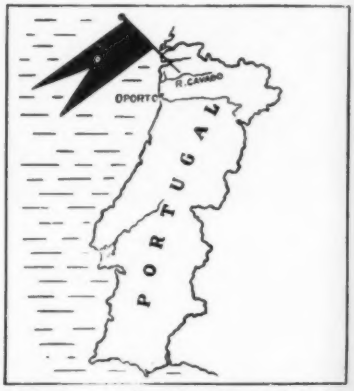
A SOUTH DURHAM STEEL PIPE LINE FOR

Cavado Vila Nova

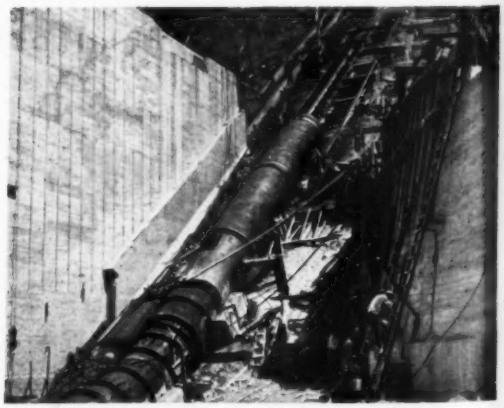
HYDRO-ELECTRIC SCHEME

Portugal

This important scheme to increase civil and industrial power resources in N.W. Portugal now approaches completion. The 300 ft. high Venda Nova dam across the River Rabagoa near its confluence with the Cavado, forms the reservoir from which water flows 1½ miles through a tunnel through intervening mountain to a valve house at the tunnel outlet. From this point a South Durham Steel pipe line carries the flow some 2,950 ft. down to the turbine house.



The South Durham steel pipe is in diameters 93", 87", 79", 32" and is electrically welded throughout with spigot and socket jointing for welding in situ. The pipe thickness is $^{13}/_{16}$" to 2$\frac{1}{8}$" for a working pressure at Power House of 590 lbs. per square inch. Total weight of the pipe line is 2,000 tons and all pipe and special pieces were erected, welded and tested by South Durham engineers and skilled workmen.



SOUTH DURHAM STEEL PIPES

South Durham Steel & Iron Co. Ltd., Pipe Department, Stockton-on-Tees. Telephone: Stockton-on-Tees 66117.

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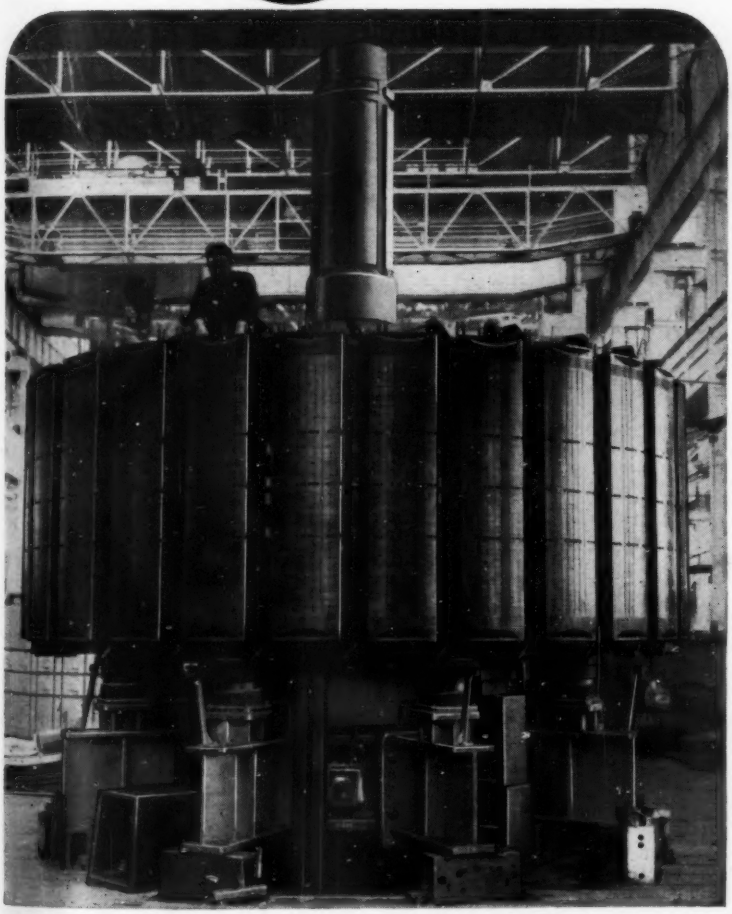
Hydro-electric equipment



A natural source of perpetual power lies in the upland lakes and rivers of almost all countries. To harness it, BTH supply all plant required for hydro-electric power schemes.

Top: Artist's impression of the Los Peares Power Station, North-west Spain.

Left: Rotor (under construction, in the BTH works) for one of the three vertical waterwheel-driven alternators (62,400 kVA., 11,000 volts, 3-phase, 50 cycles, 214 r.p.m.) to be installed in the above Power Station.

For all plant required for Hydro-electric Power Schemes — **Consult BTH**

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CLAERWEN DAM

for the

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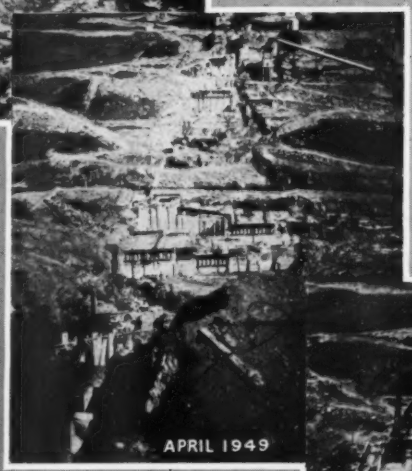

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AT LOCH SLOY, the most modern hydro-electric power station in the world, all turbine, generator, switchgear controls and meters necessary for synchronising and efficient loading have been centralised in the control room by means of *Standard* Miniature Direct Wire Remote Control, the first system of its kind to be applied to the control of hydro-electric generators.

Using entirely telephone-type apparatus, this advanced control system convincingly demonstrates that miniature, easy-to-handle equipment is safe and reliable when applied to the control of the largest generators and is indicative of *Standard's* ability to meet, in the field of telecommunications, the most exacting requirements of modern power generation.

STANDARD Generator Control desk and mimic diagram at Loch Sloy power station provides full facilities for direct indication and control of four 32 MW generators and six 132 kV feeders. Generator controls include Raise-Lower speed, Raise-Lower excitation and Open-Close generator circuit breaker. Fingertip controls operate 50V D.C. circuits at a few milliamps, enabling telephone cable to be used instead of the multi-core V.I.R. cable needed with older systems. All protective devices are continuously monitored and indicated by a STANDARD Direct Wire Alarm system.

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Our transformer department supplies A.C. transformers for outputs up to 100,000 kVA. On passing over to 3 single phase transformers the output of a group is 200,000 kVA under the same conditions.

Owing to research work on insulating methods and especially to the study of tension impulses, Oerlikon succeeded to improve the arrangement of the windings and to increase considerably the security in case of thunderstorms.

Besides transformers for great networks we supply special designs for the most various purposes, such as fire-damp-proof transformers, high tension testing transformers, locomotive transformers of concised construction, transformers for electric furnaces, etc.

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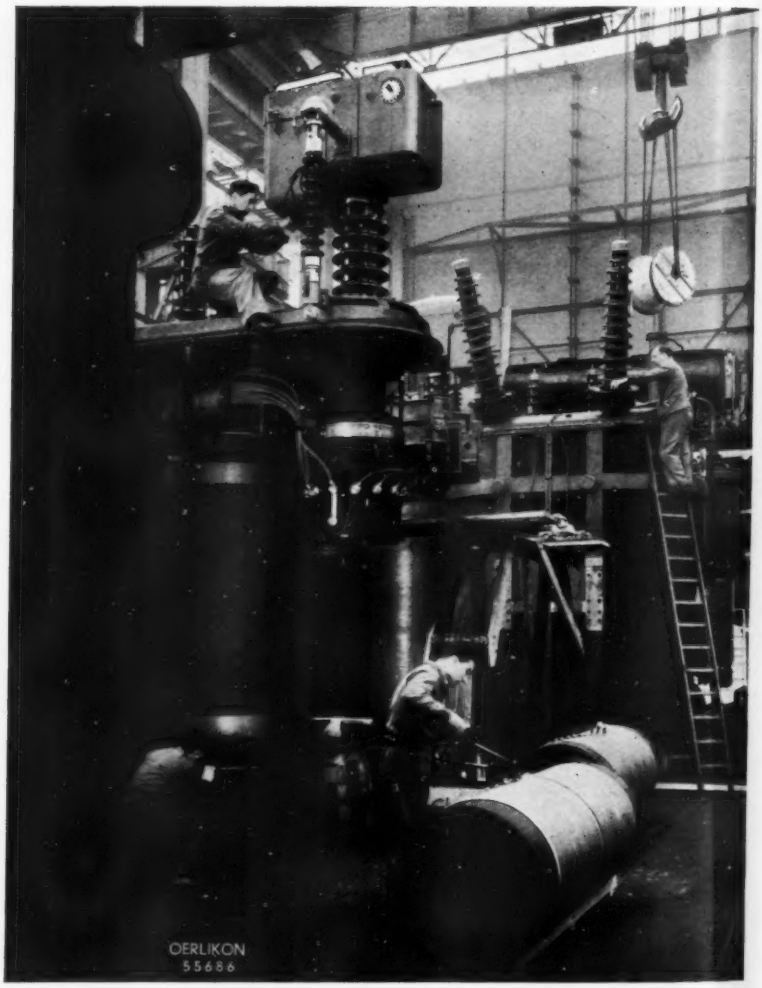

Mounting of 3 Oerlikon single phase transformers to be regulated under charge, for the Bernese Power Stations Ltd. Output per group 50,000 kVA, primary voltage 150 kV, secondary voltage 50 kV, 50 cycles.

OERLIKON ENGINEERING COMPANY, ZURICH 50 (SWITZERLAND)

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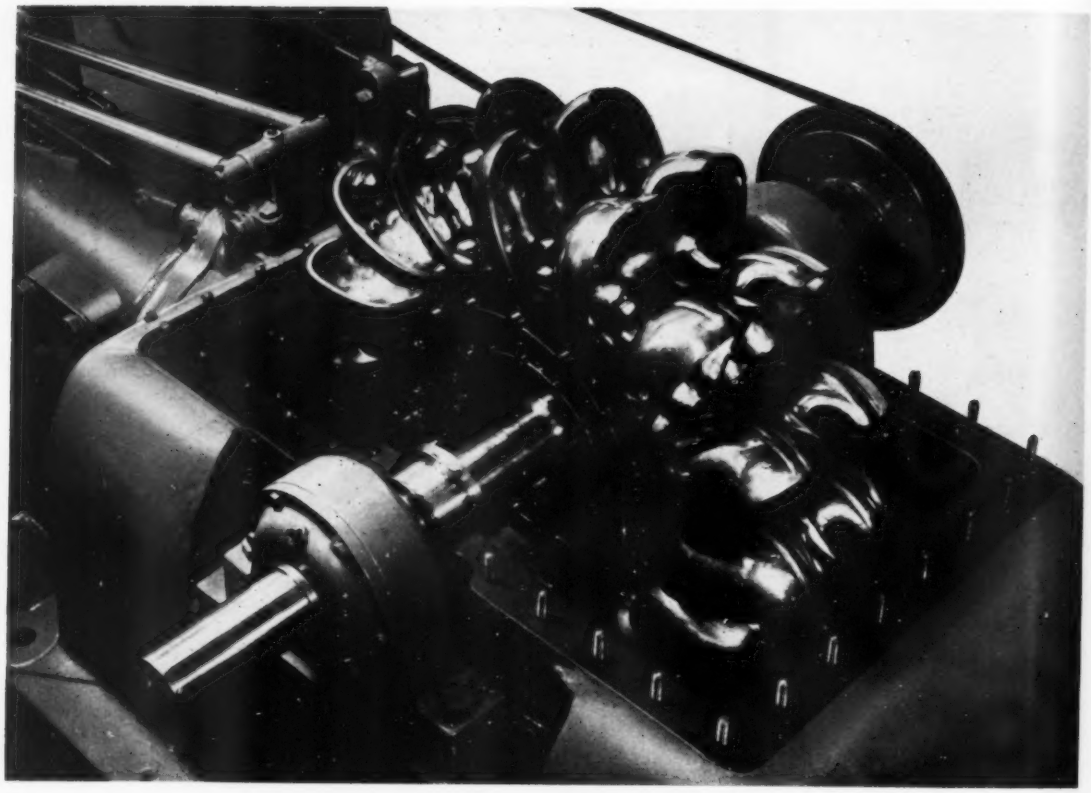

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A Pelton turbine which has been designed for installation at a tea-factory in Ceylon. The output of this machine is 70 B.H.P. under a net head of 200 ft., and it is arranged as a single running unit with an oil pressure governor actuating deflector and spear control mechanism.

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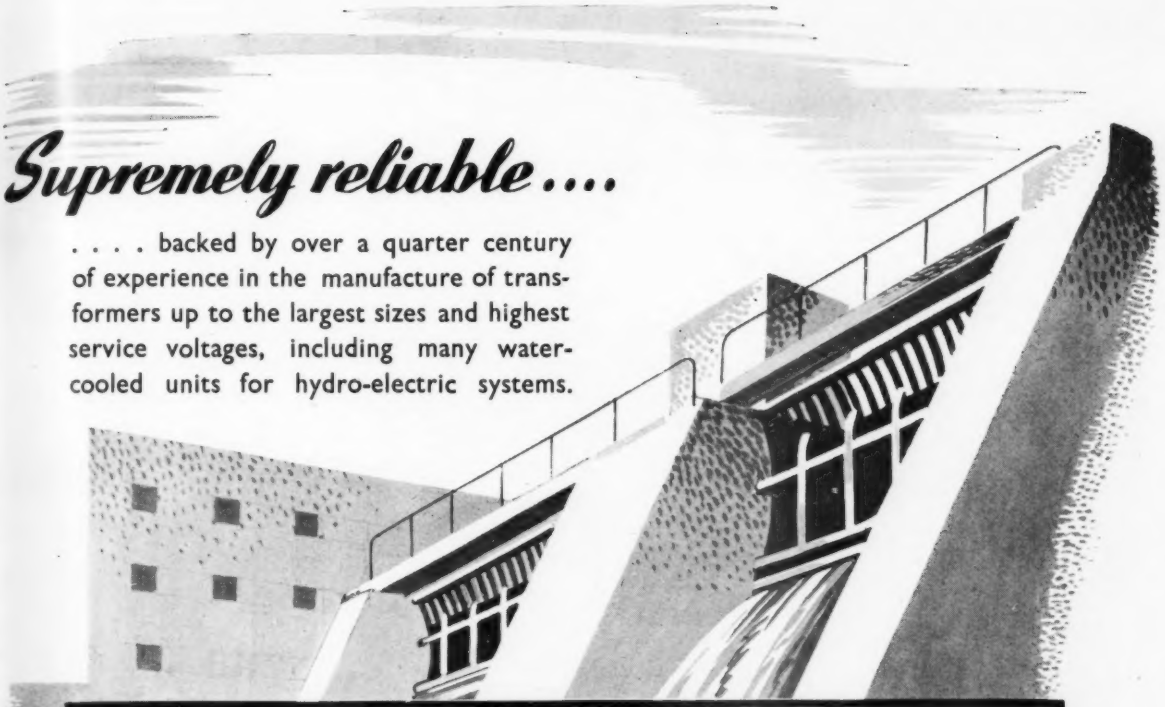
. . . . backed by over a quarter century of experience in the manufacture of transformers up to the largest sizes and highest service voltages, including many water-cooled units for hydro-electric systems.



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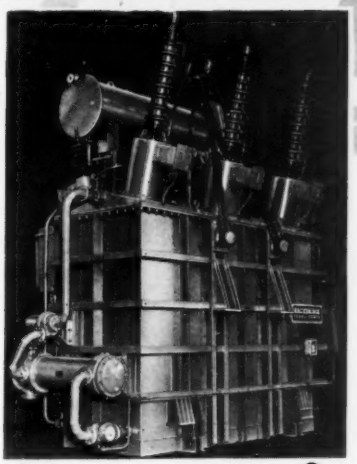

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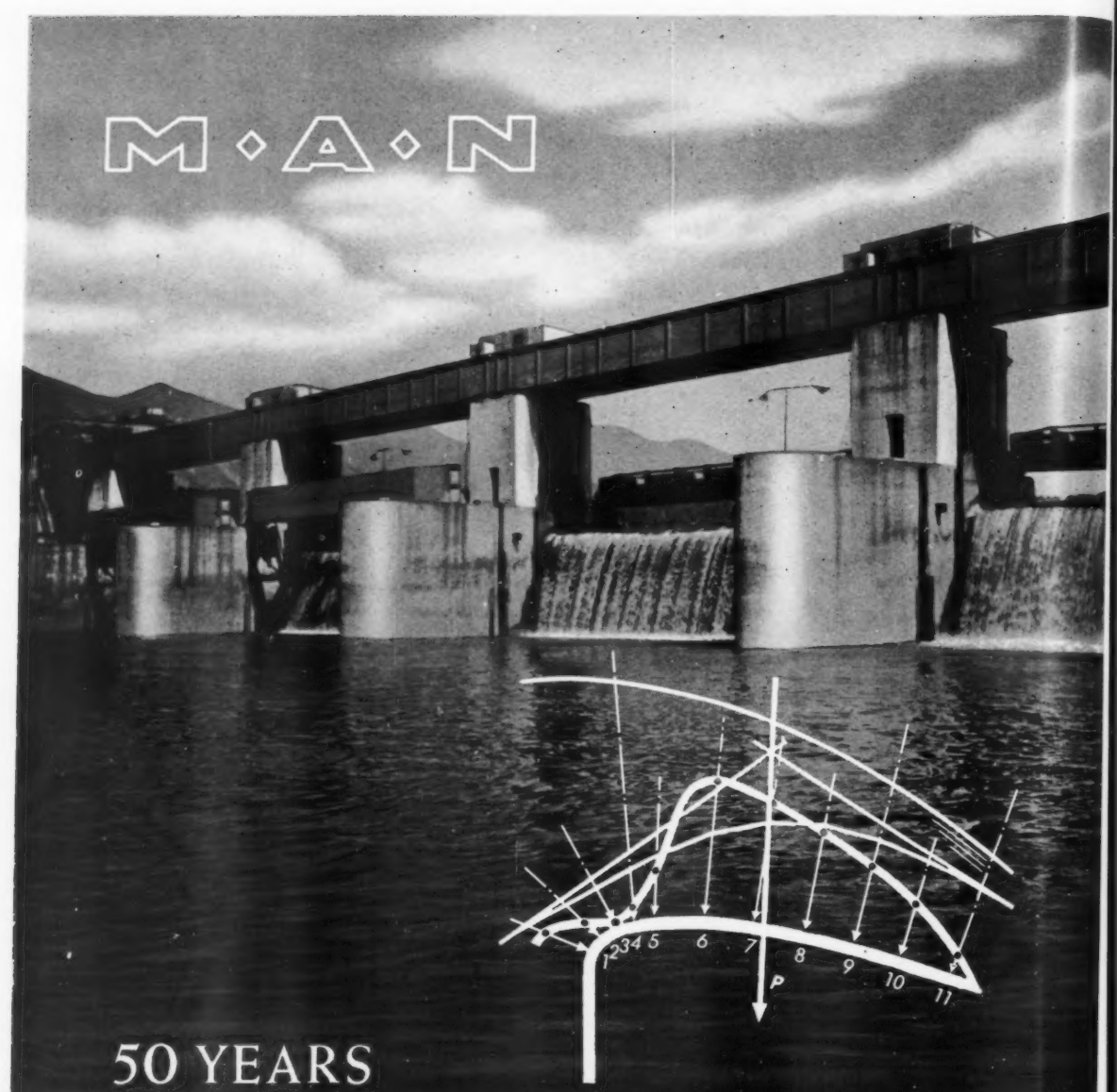

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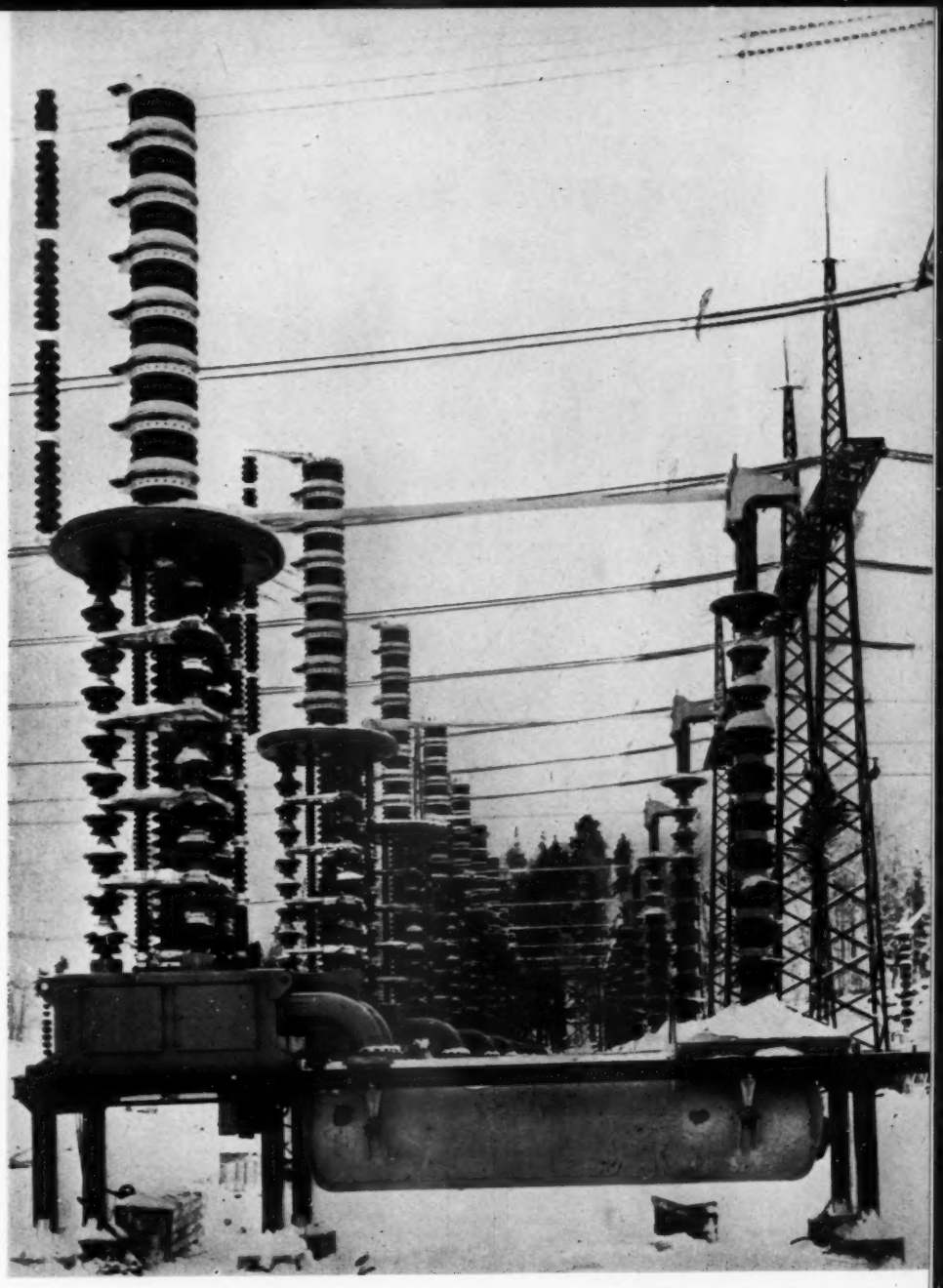
MASCHINENFABRIK AUGSBURG-NUERNBERG AG., GUSTAVSBURG WORKS, GERMANY

The new power line includes five 380 kV air-blast circuit breakers, all supplied by ASEA. The photo shows three of these breakers.



The first 380 kV equipment in the world is now in service

During the week March 30th—April 6th this year, the first Swedish 380 kV power transmission line was taken into service—the first in the world to be operated at this voltage level.

ASEA has delivered the switchgear equipment and transformers required for this system, including nine single-phase transformers, rated at 100,000-115,000 kVA and five three-phase air-blast circuit breakers with a breaking capacity of 8,800 MVA (B.S.S. rating), as well as lightning arresters, instrument transformers, isolators, etc.

The ASEA Cable factory has supplied 380 kV cables connecting the underground transformers with the overhead power line.

380 kV transformers delivered by, or on order from, ASEA represent an aggregate power of 3,000,000 kVA.

In addition to supplying the bulk of the equipment for the new power system ASEA has carried out much of the preparatory research work in collaboration with the Swedish State Power Board.

VÄSTERÅS — ASEA — SWEDEN

OFFICINE GALILEO
DI BATTAGLIA TERME (PADOVA) S. p. A.

Barrage at Bassano del Grappa—4 Sluice gates, 16 metres long, 4.6 metres high, of which two are automatic

SLUICE GATES
CRANES
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AND ALL TYPES OF STRUCTURAL STEELWORK & GENERAL ENGINEERING

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- more than 100 years' experience.
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GALILEO WORKSHOPS OF BATTAGLIA TERME S.p.A. PADUA, ITALY. PHONE: PADUA 25689-26003 GRAMS: WORKSHOPS, BATTAGLIA TERME, ITALY

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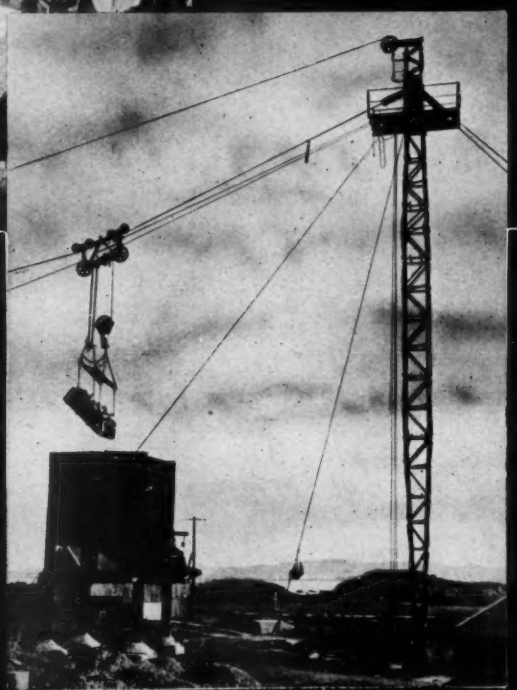

Giant hydro-electric schemes in all parts of the world make use of BRECO Aerial Cableways and Ropeways for the transportation of vital materials.

The largest builders of heavy capacity Cableways and Ropeways in the world, BRECO will supply advice and technical information willingly, and without obligation.

BRECO *Aerial* ROPEWAYS

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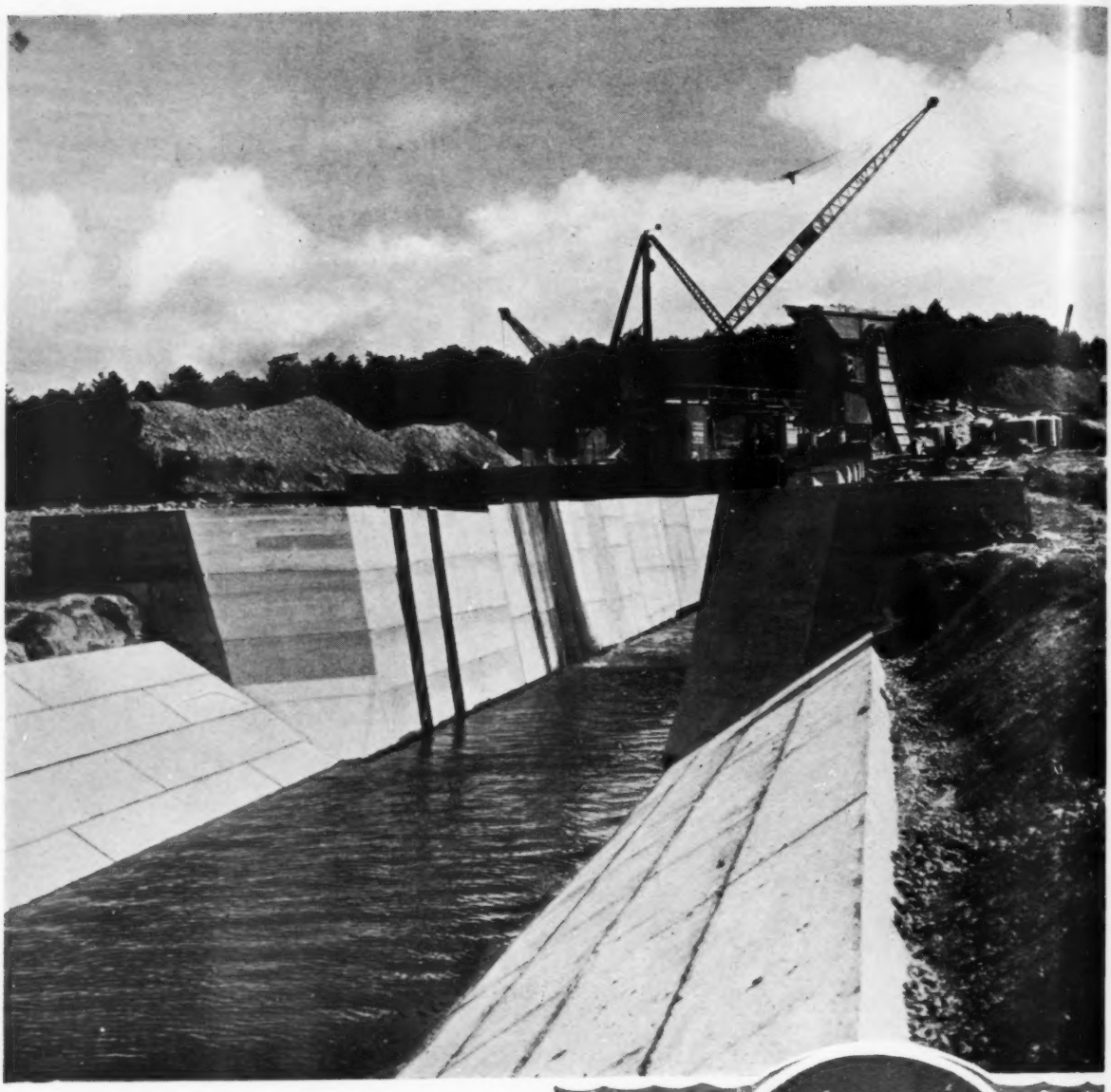

Work on the diversion tunnel and channel at the Weir Wood Reservoir Contract, which is one of the water conservation schemes being carried out by the Company

Consulting Engineers: Herbert Lapworth Partners

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COSTAIN
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The Vevey speed governor

administers the correct dose

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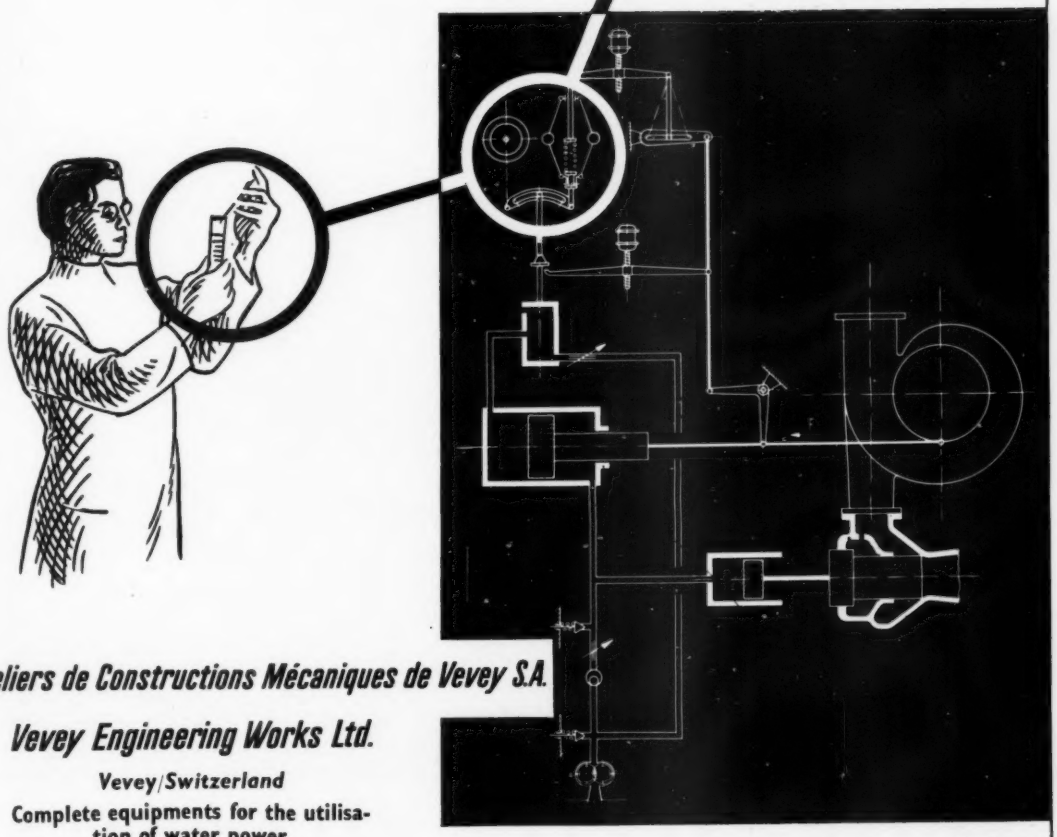

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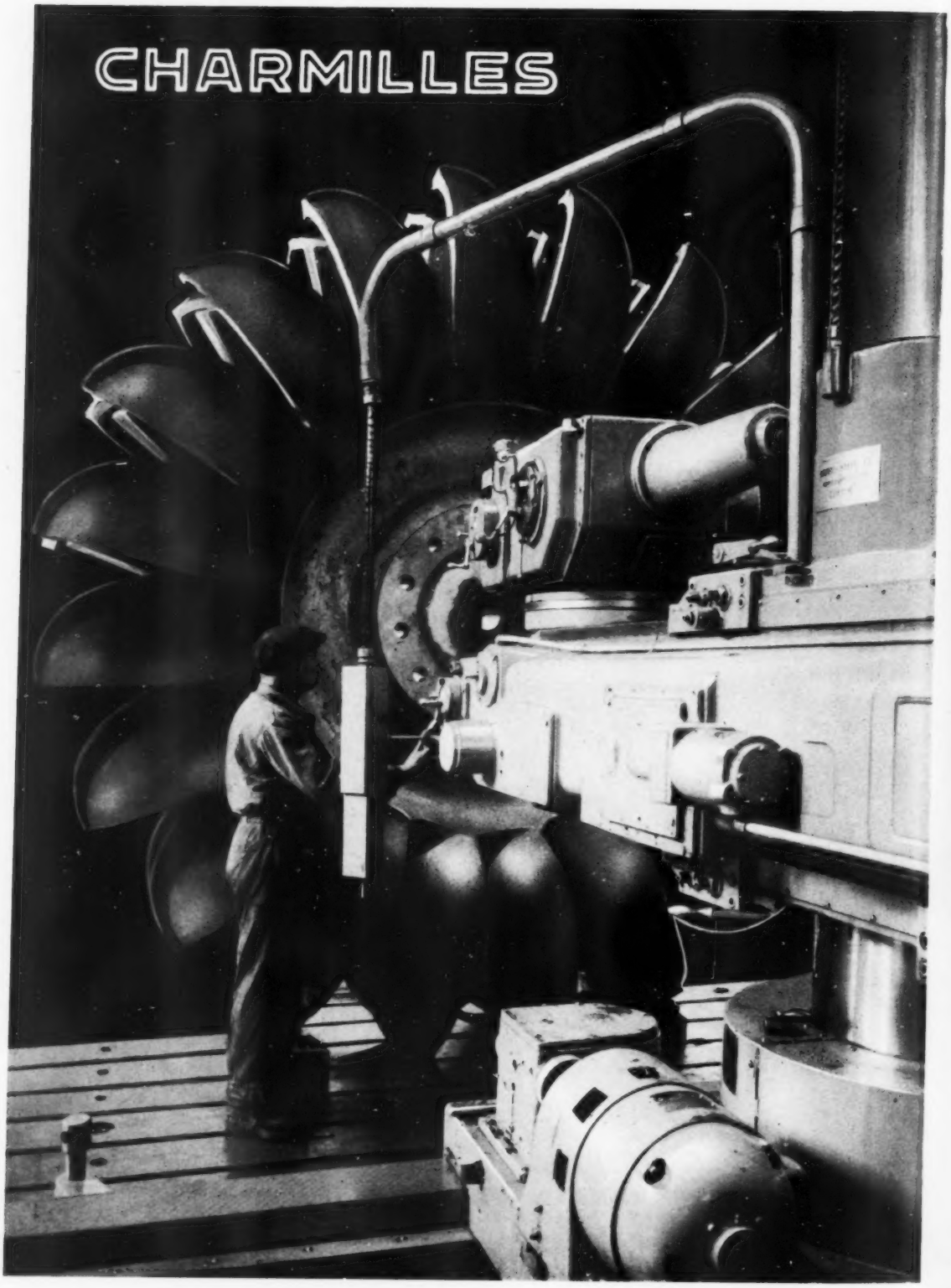

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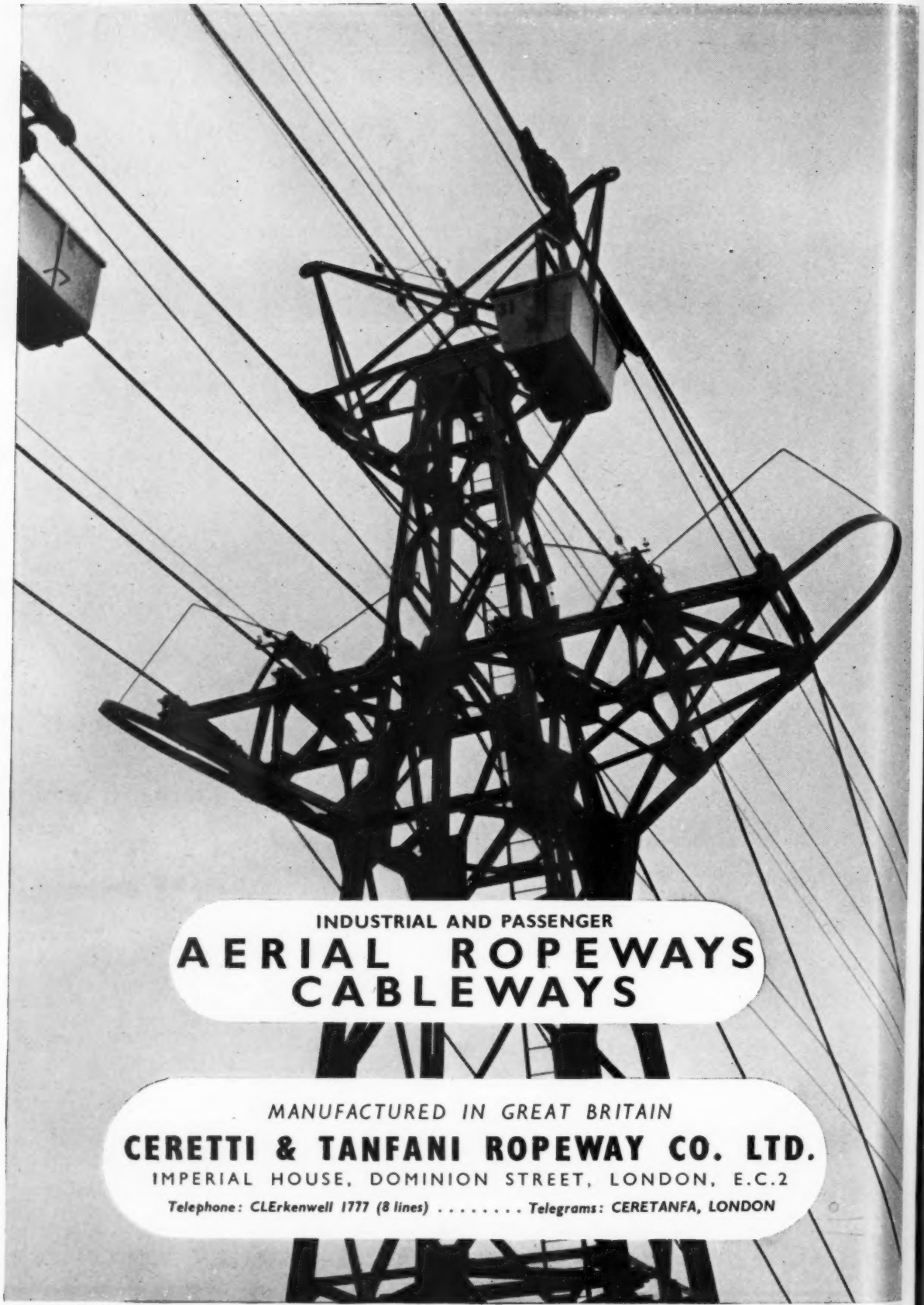
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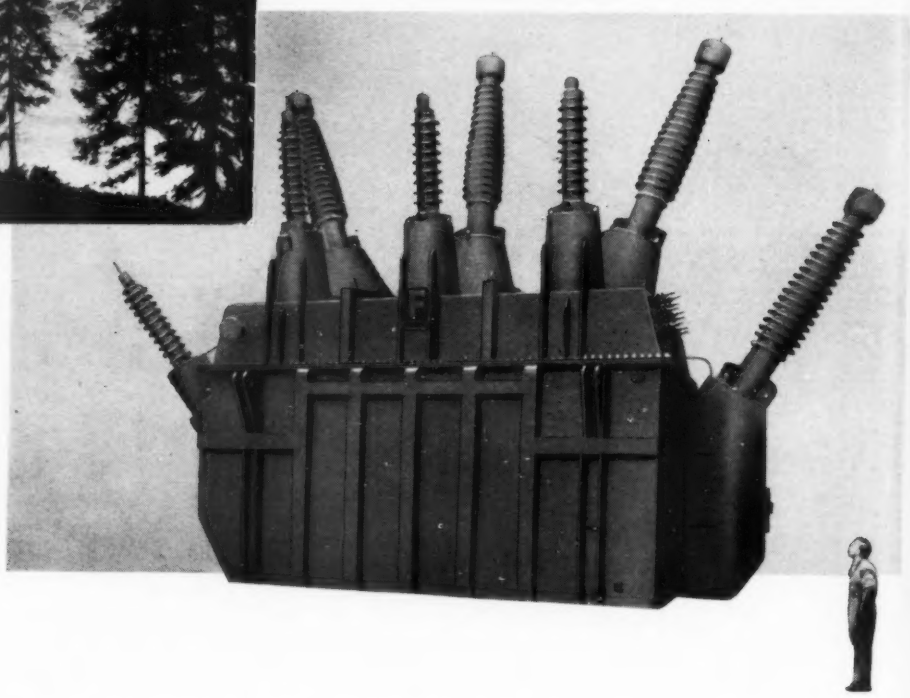

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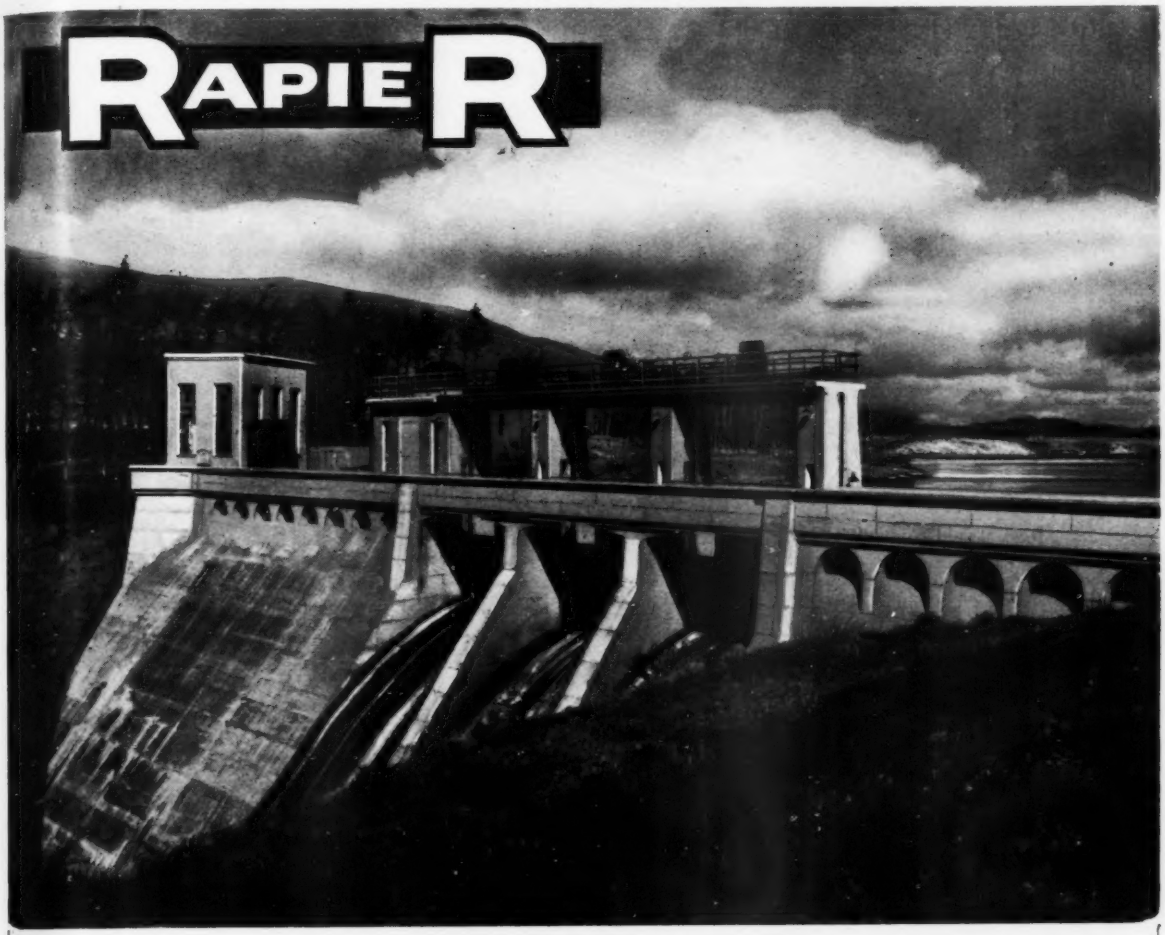

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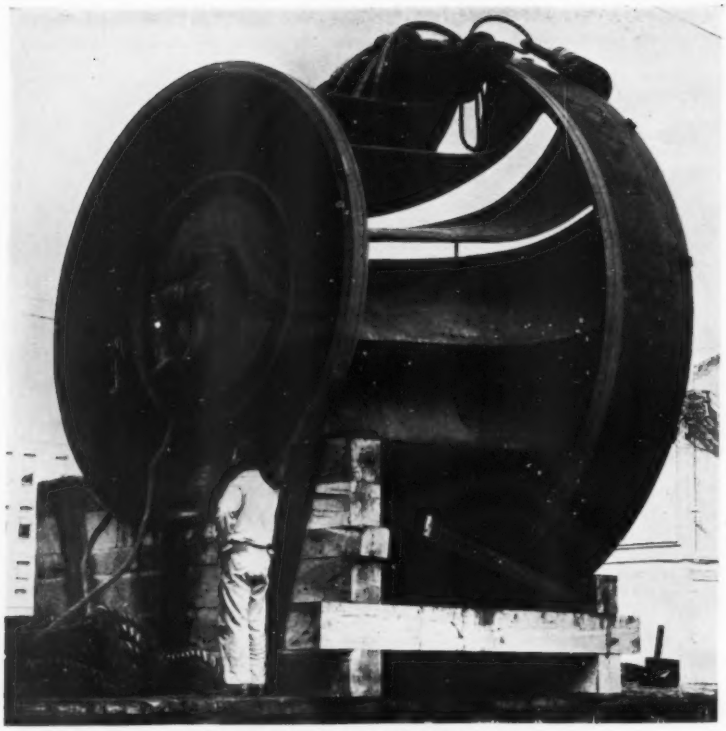

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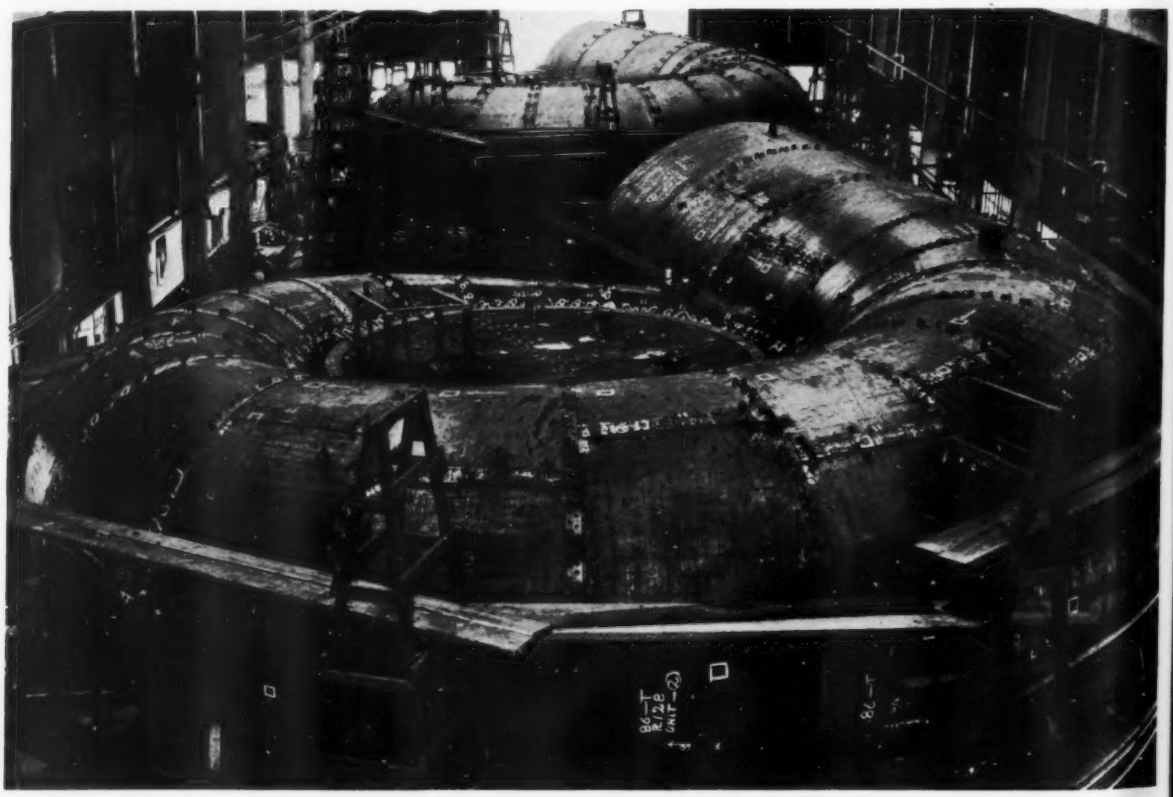

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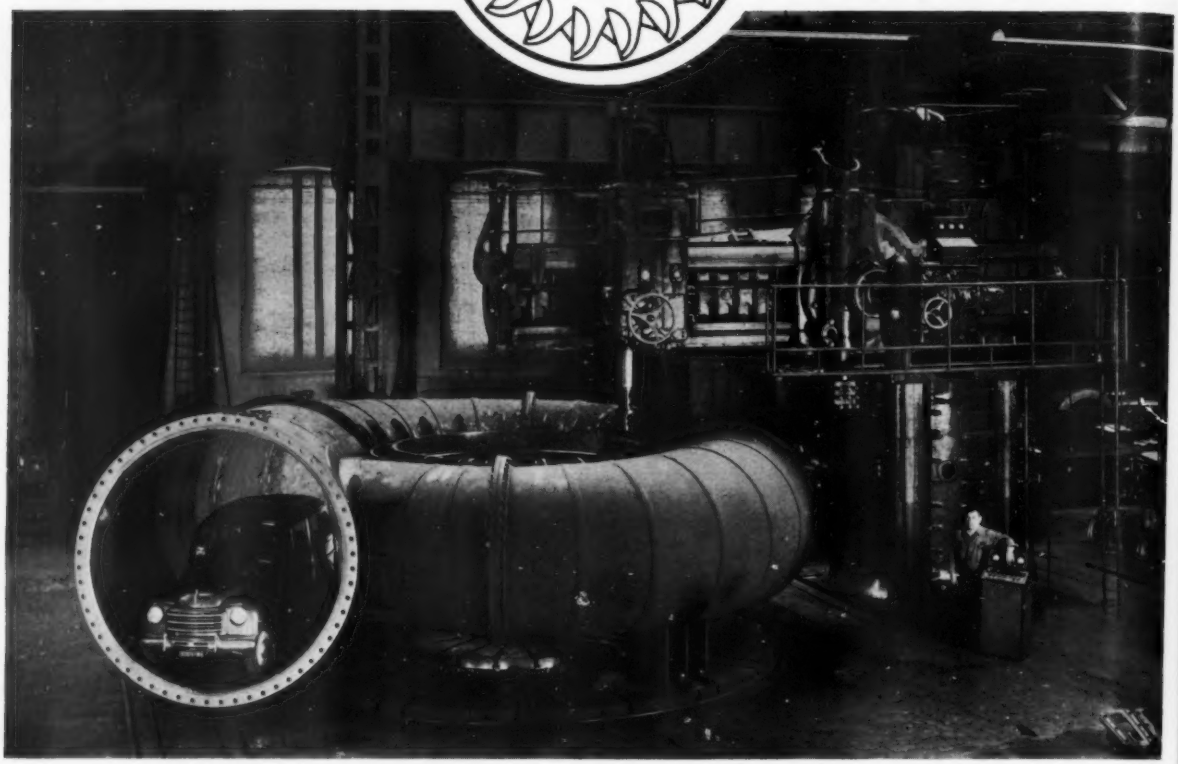

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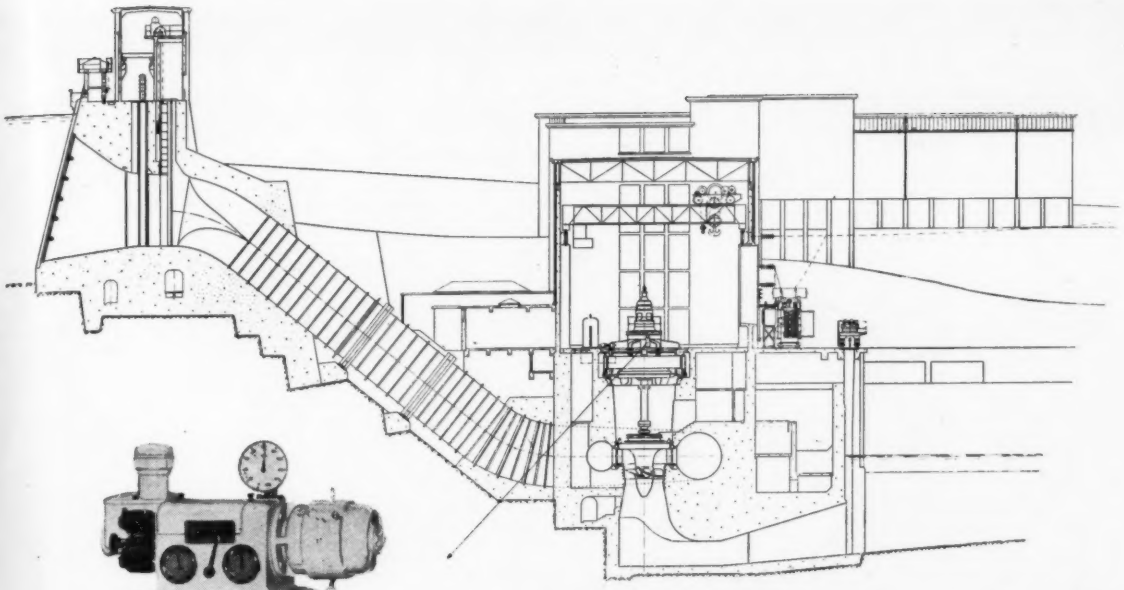

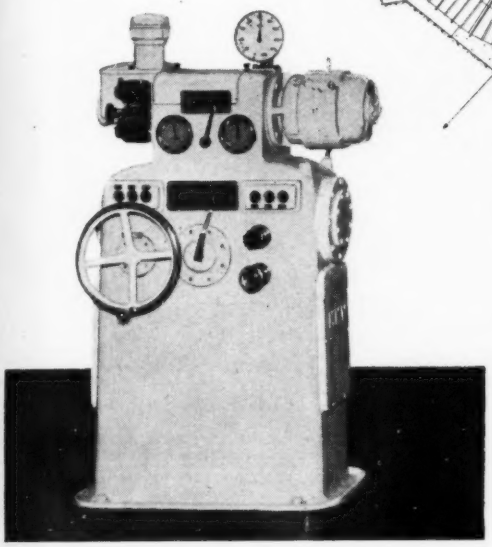

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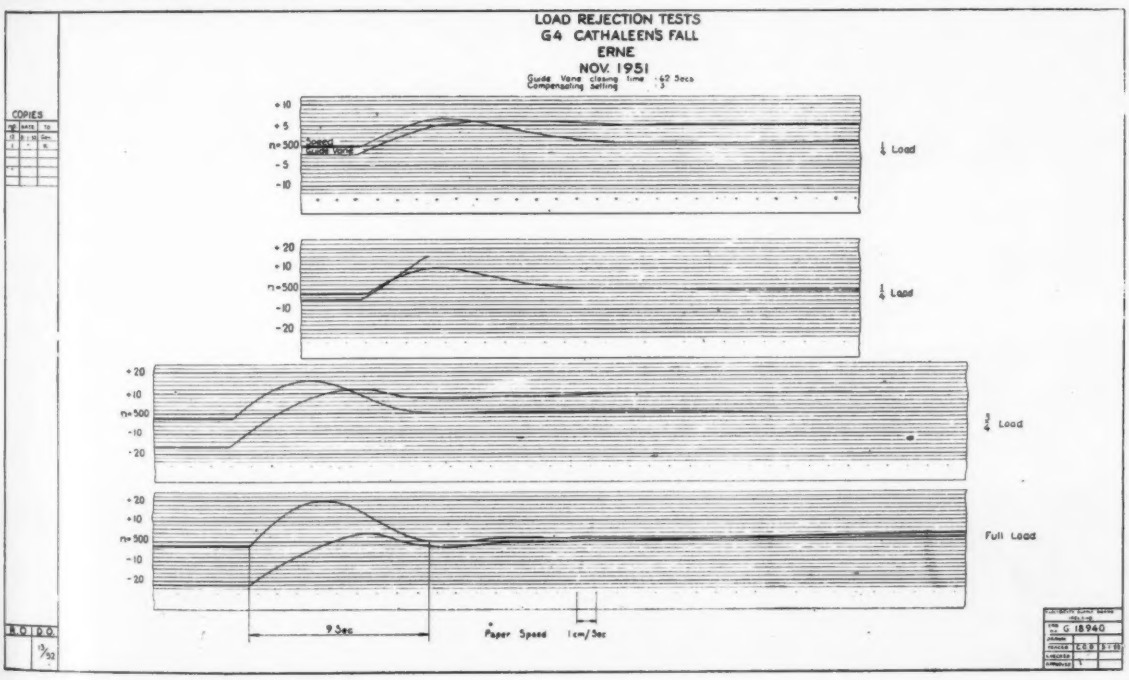

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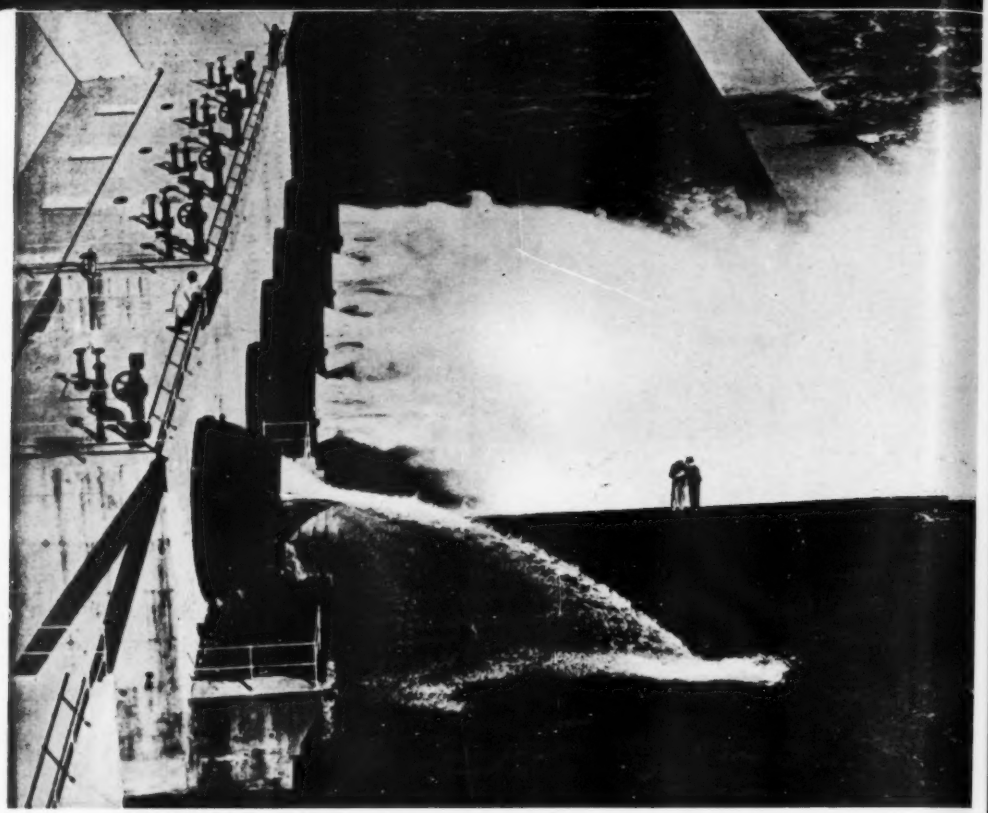

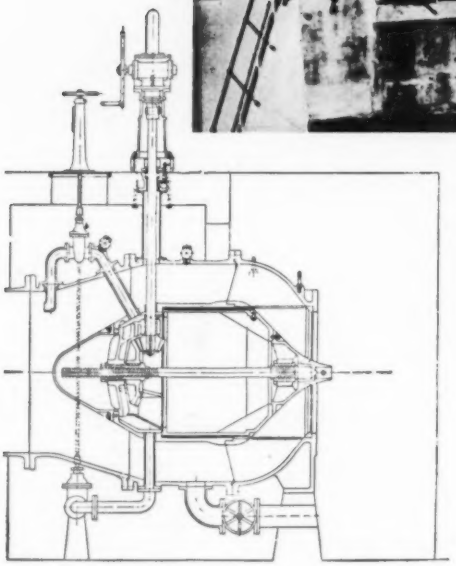

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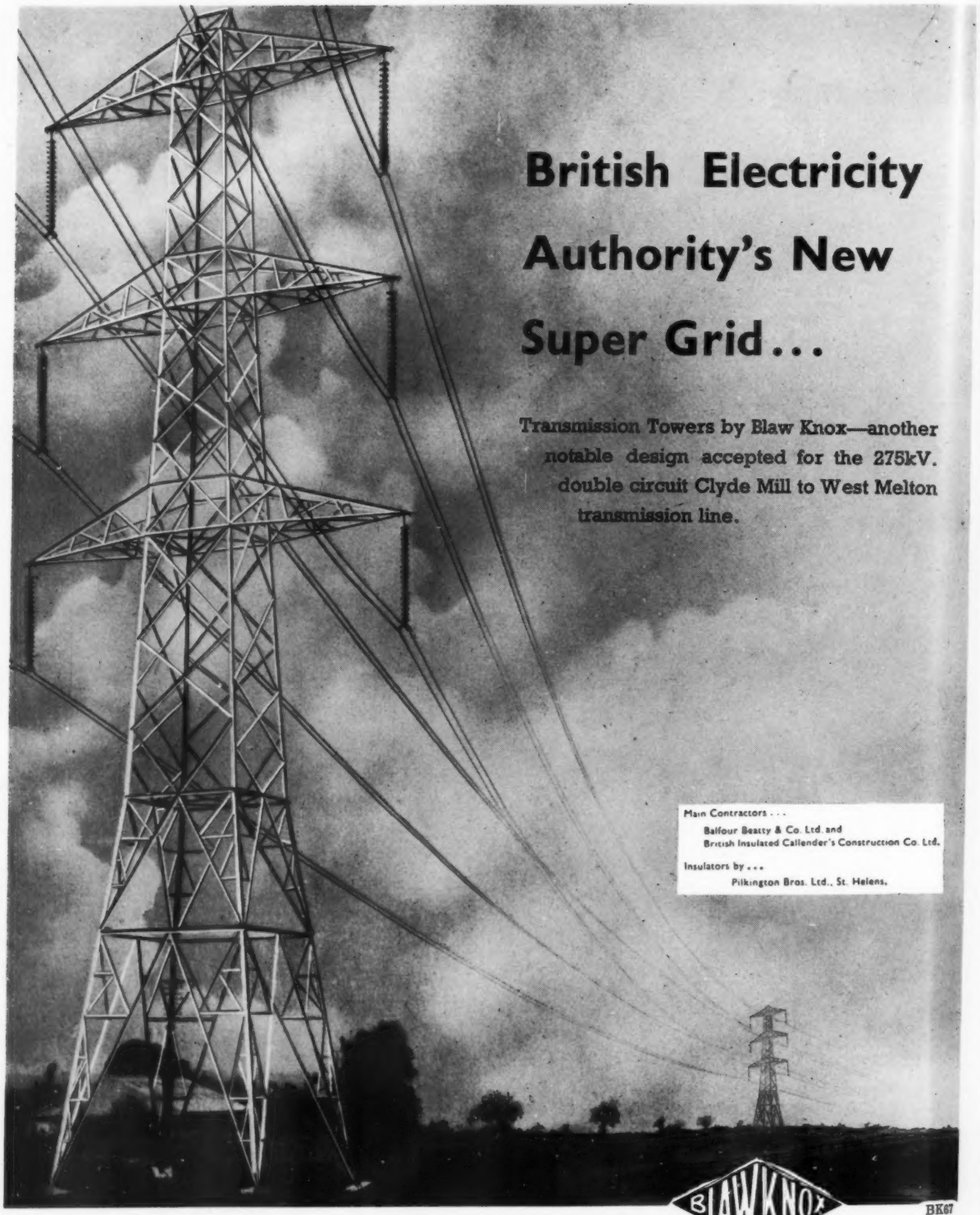
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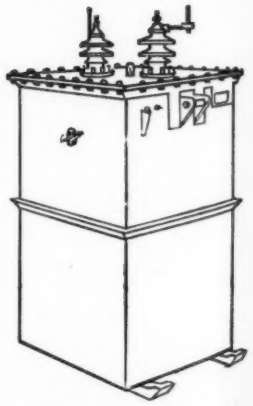

Further information on the use of capacitor banks is contained in Publication No. 278V *available on request.*

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Control of the Weather

IN recent years considerable interest has been displayed in the possibility of influencing the weather by human intervention. Hydro-electric engineers in many countries are perennially faced with the possibility of long droughts, which, in view of the character of the rivers on which their plants rely, may rapidly result in empty reservoirs and consequent load shedding or complete shutdown. France, Portugal and New Zealand are countries where recent drought experiences have left unfortunate memories. Engineers in those parts of the world may well display much more interest in rainmaking than those who are concerned with terrains where the presence of lakes and the small seasonal differences in flow mean that artificial rainfall would be a source more of embarrassment than of benefit.

Two basic facts about rainmaking must be borne in mind: first, rain can only be caused on occasions where water-bearing clouds pass over the area; and secondly, it is still not possible to arrive with certainty at an evaluation of any particular rainmaking operation.

The operators who have done most in this field use silver-iodide crystals as artificial nuclei on which raindrops can form. It is claimed that the projection of large numbers of iodide particles into a cloud will cause it to precipitate its moisture under conditions of temperature, vapour pressure, dewpoint and barometric pressure which would not in themselves have caused a fall of rain.

The silver-iodide, or other similar substance, is projected into the cloud either from the air or from movable projectors on the ground. The aerial method is expensive, and it may well happen that flying conditions are impossible at a time when there is an urgent need to operate. On the other hand, aerial "seeding" can be carried out anywhere, at almost any normal cloud height: and in this way, for example, rain can actually be "prevented" on a given area by causing clouds to precipitate at some point on their path before the area to be protected is reached. Those who favour projection from the ground say that with the aid of extensive local meteorological knowledge they can manipulate their "guns" in such a way that rising air currents from mountainsides and hills will direct the constant stream of seeding particles into the clouds with an even greater certainty and intensity than is the case with aerial spraying.

Any rainmaking operation must, of course, affect many interests other than those directly concerning the hydro-electric engineer. Obviously the agriculturalist is vitally interested; and it is indeed the farming community who have contributed most, in the American experiments in rainmaking, by way of continued financial support to the investigators. Reports from different parts of the States give conflicting views on the success of the operations, but a number of agricultural interests have been sufficiently impressed with two or three years' results to continue, and even extend, their support. On the power generation side, at least one large power corporation has engaged the services of the rainmakers on a long-term basis. The official meteorological authorities and the State Departments concerned with agriculture and with aviation have so far not endorsed with official approval and support the creation of artificial rain.

It will be obvious that rainmaking, however successful it may become, is closely related to accurate forecasting. Rainmaking generators cannot be continuously in action, even for days and certainly not for weeks; and so the meteorologist must be able to indicate with a high degree of certainty when particular cloud formations are likely to be over the desired rainfall areas, and he must be able to forecast the other determining factors—temperature, barometric pressure, and so on—with equal accuracy. Otherwise, the operation takes on a hit-and-miss nature, and can scarcely be economic.

Thus rainmaking may well play an important part in future hydro-electric schemes, when weather forecasting has become truly reliable. It is possible to visualise a situation where the engineers in the central control room of a large interconnected hydro-

electric power system, having charge of not only power generation but also water control for irrigation, navigation and domestic supplies, will have an electronic computing machine which will take minute-by-minute account of a vast amount of meteorological data, transmitted continuously from hundreds of unattended weather stations; and will then, entirely automatically, seed the clouds to precipitate rain when it is needed.

Concurrent with possible improvements in artificial rainmaking must come the development of appropriate legislation and control. Rainfall may be greatly desired by one section of the population when another, equally large, is extremely anxious to keep the rain away — for example, the hundreds of thousands of people who, on a given Saturday, may be attending open-air functions. This disparity of interests could only be resolved by firm control, and even then it is possible to foresee some involved legal battles if the promoters of today's outdoor sports events claim that the rain which reduced their revenue was caused by yesterday's seeding operation carried out at the request of the Hydro Board to fill the reservoirs! It is understood that in America the rainmakers, on at least one occasion, have been endeavouring on the one hand to claim a fee from their sponsors for a successful operation, and at the same time to claim total failure in view of a heavy law suit against them for interfering with outdoor festivities. It has for centuries been the custom to pray for rain; soon, perhaps, to pay will be enough!

Hydro-Electric Potentialities in Papua and New Guinea

GEOLOGISTS and engineers and other interested parties have given the potentialities of the territory of Papua and New Guinea some consideration and stress the immense hydro-electric potential, which dwarfs Australian possibilities. The mountains run up to 15,000 ft., the water-flow gradients are marked, and the rainfall is heavy, averaging over 100 in. a year. In the vicinity of Port Moresby the Department of Works and Housing anticipates that 250.000 h.p. can be generated on the Laloki river for a capital expenditure of £3¼ million. Anticipated capital expenditure on the Kiewa scheme in Victoria, of virtually the same horsepower, is £25 million.

New Zealand Supply

ALTHOUGH 13 per cent. more electricity was generated in the North Island of New Zealand in the year ended March 31, the demand for power was met only by loss in storage. This amounted to one foot at lake Taupo, and 17 ft. at Waikaremoana. The generating manager of the State Hydro-Electric Department considered that a reasonable demand in the North Island this year could be met by providing an allocation of 7½ per cent. above last year. This of course depends on rainfall being at least as much as in 1950-51, and on Maraeki supplying 30,000 kW by the end of October and 108,000 kW three months later. In the South Island storage is good. All the lakes are full except lake Mahinerangi which supplies Waipori. It is expected that the power generated will amount to 987 million kWh, 7 per cent. more than last year. This amount is exclusive of the Nelson-Marlborough area.

Progress in Puerto Rico

A CONSIDERABLE amount of money is now being invested by the Puerto Rican authorities in hydro-electric development. In 1949 the Government announced that 24 million dollars were to be spent in irrigation and power development work in the Lajas Valley and we now hear that this project is proceeding satisfactorily. It is understood that the completion date, scheduled for 1955 will, on present progress, be kept. American capital to the extent of eight million dollars has been provided for Puerto Rican developments which have as their ultimate aim the doubling of the hydro-electric output of the water power plants in the country from the present figure of about 250 million units a year.

A Venezuelan Development

AS an indication of the amount of time required for the preparation of plans for a hydro-electric project, it may be of interest to know that the first mention of a possible hydro-electric development on the Uribante river in Venezuela came early in 1949, when the Venezuelan Development Corporation commenced a preliminary survey. In 1951, completed plans were passed and it was then thought that a generating station on the Uribante river could have a capacity of 20 MW. The news that specifications and tenders are now going out to manufacturers for a station containing two 13·5 MW units as a first stage, with a third unit to follow, comes three years after the commencement of planning—a period which any experienced hydro-electric engineer would agree is not too long for a project of this kind, and a period, moreover, which he could easily defend, if uniformed criticism suggested that it might have been completed much more rapidly.

Five-Year Plan for Japan

LIKE most countries, Japan has experienced a chronic power shortage during the postwar period, and in an attempt to devise remedial measures the Public Utilities Commission has formulated a five-year plan of development and undertaken a reorganisation of the electric power industry. The rational development of water-power resources is a first and indeed main objective, and emphasis is placed on the large-scale reservoir type of plant. In broad outline the plan envisages the installation of 6 million kW of additional hydro-electrical capacity and 1·32 million kW of thermal plant. Of this 4·23 million kW of hydro and 1·23 million kW of thermal plant are expected to be in commission by the end of the fiscal year 1955. These schemes are estimated to cost a total of 706·8 billion yen for the five-year period, involving the consumption of 1,400,000 tons of steel, 167,000 tons of copper (for which some aluminium may be substituted) and 104 million tons of cement.

Surveys are being conducted on the Tadami and Kumano rivers, and engineers of Overseas Consultants, Inc., are now engaged in planning the actual constructional work.

At the end of 1951 the total generating capacity (hydro and thermal) installed in Japan was 10,540,000 kW and to this total hydro power is expected to add

243,193 kW of capacity during 1952, 1,146,990 kW during 1953, 1,456,955 kW during 1954 and 1,148,090 kW during 1955. Thereafter hydro-electrical development is expected to proceed at the rate of about 1¾ million kW additional capacity per annum. The total water-power resources of Japan, including those already developed, are estimated to amount to 20 million kW. The provision of the necessary plant is regarded as being well within the capacity of Japanese industry, which even in 1939 had achieved a production rate of 1,220,000 kW of waterwheel generator capacity.

Damodar Valley

NEWS from India that the Damodar Valley project is keeping up to schedule and that the first dam on the Barakar river will be completed next month is of special interest in view of the somewhat gloomy view of the progress of this project taken in its earlier years. It was started in 1948 and Dr. Sudhir Sen, for example, the secretary of the Damodar Valley Corporation, in December 1950, wrote at great length in one of the leading Indian organs of opinion about the difficulty they had experienced in obtaining sufficient skilled labour. He pointed out that India had theoretically sufficient technicians, but that they lacked length of experience and the opportunities for participating in hydro-electric work of all kinds which was possible for similar technicians engaged in hydro-electric projects in most of the Western countries.

The Damodar Valley project in West Bengal, when complete, is designed somewhat on the lines of the Tennessee Valley Authority for complete water control, and will influence the irrigation of 967,000 acres of land, while the incidental power stations will generate 240 MW.

Tennessee Valley Progress

DETAILS of the Tennessee Valley Authority's continued development are given in the 18th annual report of the Authority covering the fiscal year ended June 30, 1951. Good progress was made towards doubling the capacity of the power system to more than 6,800 MW in 1954. It was estimated that the energy requirements of the Federal agencies would increase by 9,500 million kWh within three years, that those of the municipal and co-operative systems would increase by 4,200 million kWh and those of the large industries served directly by the Tennessee Valley Authority by 3,300 million kWh.

At the close of the fiscal year, 3,632 MW of new capacity was under construction or planned. The Authority had five steam and hydro-electric generating plants under construction for a planned capacity of 2,790 MW. Work on two multiple-purpose dams—Boone and Fort Patrick Henry—was in progress, involving 111 MW of capacity. There were plans for the installation of new units in seven existing T.V.A. dams involving a total of some 325 MW.

The installed capacity of the system as a whole was increased by 187·5 MW during the year. The Authority placed in operation the South Holston dam on the South Fork of the Holston river, with an installed capacity of 35 MW. With over 625,000 acre-feet of controlled storage, the dam helps to reduce the risk of serious flooding damage at the industrial city of Kingsport, Tennessee, besides contributing to the control of major basin-wide floods.

Of the 18,000 million kWh generated during the year 86 per cent. was obtained from hydro-electric plants and 14 per cent from steam stations. In addition, the Authority obtained 2,000 million kWh from other systems. Water conditions during the year were generally good. The Authority sold 16,500 million kWh, of which 70 per cent. went to the so-called "preferred" customers—municipal and co-operative systems and Federal agencies.

The completion of the South Holston dam brought to 28 the number of major dams controlling the Tennessee river and its tributaries.

High-Voltage Direct Current

FROM time to time in this journal we have touched on the trend towards the use of high-voltage direct current for the transmission of power from remote hydro installations to the centres of industry. The Swedish pioneer scheme involving the 62½ mile sea crossing from the mainland to the island of Gotland has sharpened interest in other manufacturing countries in this method of power transmission, and a paper read before the Institution of Electrical Engineers in April by Dr. H. von Bertele, previously Chief Rectifier Engineer of Siemens in the prewar years, on the design of high-power mercury-arc converters drew a very large audience.

The paper described the author's researches into many design details of large mercury-arc valves operating at voltages of the order of 250 kV and having a capacity of 1,000 A.

In the discussion, Mr. C. Ehrensperger of Brown, Boveri & Co. Ltd. of Switzerland, stated that his firm was ready to build the converting equipment for a d.c. transmission system and to share part of the financial risk involved in the development work for such an enterprise, providing a power company or authority would co-operate.

British Industries Fair, 1952

IT was a little disappointing that the Engineering Section of the British Industries Fair this year—held as usual at Castle Bromwich, near Birmingham—did not contain more than a single stand in which hydro-electric plant was directly featured. This stand was that of the British Thomson-Houston Company, who showed a full size model of one of the 62,400 kVA alternators for the Los Peares station in Spain. Nevertheless, very many stands indicated remarkable advances made during the year in the design of equipment concerned with the constructional side of hydro-electric projects and the auxiliary plant needed in the power station.

Riva and Calzoni Products. A handsomely illustrated catalogue issued jointly by Costruzioni Meccaniche Riva of Milan and Alessandro Calzoni of Bologna, gives a brief description of the extensive works of these two firms, together with brief particulars and illustrations of a few of their more notable contracts, which include Pelton, Francis and Kaplan turbines, hydraulic pumps, valves, automatic weirs, regulating sluices and turbine governing gear.

Fig. 1. View of the works at the Venafro end of the tunnel

The Volturno-Garigliano Scheme

Particulars are given of this Italian project, constructed by the Società Meridionale de Elettricità, with special reference to the tunnelling work

BEGUN about one year ago, the construction of Volturno-Garigliano hydro-electric power plant was the result of extensive study by the engineers of the Società Meridionale di Elettricità (S.M.E.) of Naples, who for several years devoted their energies to the complex problems involved. The scheme is now nearing completion and is not only a notable achievement in itself but is one of the greatest importance to Southern Italy as the 131 million kWh per annum of energy that will be produced will make a valuable contribution to the industrialisation of the South.

The Volturno river has a length of 170 km. (105 miles) from its source in the mountains of Meta, in the Molise district, to where it flows into the Gulf of Gaeta. The course of the river, because of its different characteristics, can conveniently be divided into three distinct parts: the first from the source at Cape Volturno to its confluence with the Vandra torrent; the second down to its confluence with the river Calore, which is its most important tributary; and the remainder from the junction with the Calore to the Tirrenian Sea.

The power plants under construction for the S.M.E. are situated on the second portion of the river Volturno, the development of which, however, presented a number of difficult problems. The first part of the river was already developed by the Ente Autonomo de Volturno whose power stations were in operation, while the lower part serves the important function of irrigating the Campania — a fact which has so far

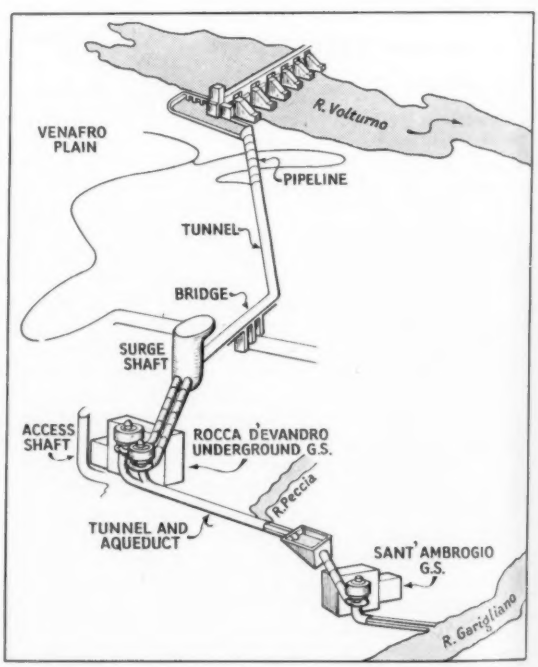


Fig. 2. General outline of the Volturno-Garigliano scheme

inhibited a definite scheme for its utilisation as a source of hydro-electric power.

The middle portion has presented the engineers with two principal difficulties; the first of these is the width of the river below the junction with the Vandra, which would have necessitated a big and expensive dam and intake works; and the second, which is still more important, is the fact that from the natural head at the junction of the Vandra river, which is 236 m. (770 ft.) above sea level, to where it joins with the Calore river at 35 m. (115 ft.) above sea level, there is only a difference of 200 m. (655 ft.) over a distance of 50 km. (30 miles). Further, of this 200 m. (655 ft.) the first 60 m. (197 ft.) is located between two points comparatively near to each other, which, while offering fairly easy utilisation of such a head, would greatly complicate the problem of utilising the remaining downstream course.

Also, the river receives several tributaries which add to the discharge and which proved a deterrent against the possible utilisation, by means of a reservoir, of this central portion of the Volturno river. In fact, every scheme which attempted to utilise the heads and discharge in the middle part of the river were negatived by the extreme lengths necessary for supply channels, which would have been altogether out of proportion to the amount of power that could be produced.

In consideration of these difficulties, the engineers of the S.M.E. have adopted a more convenient, easy and alternative solution by conveying the waters of the Volturno to the nearby Garigliano river and, in this way, utilising the different levels of the two river beds.

In 1950, in accordance with this plan—first proposed by Eng. Giovanni Battista Canevari — the S.M.E. started the construction of two power stations, one at Rocca d'Evandro and the other at S. Ambrogio, combining the waters of the Volturno river with the intake below the junction with the Sava river and discharging them out in the Garigliano river below, where this joins the river Peccia. In this way the difference in height provides a head of 160·40 m. (525 ft.) and gives a water discharge of 13·20 cu. m. per sec. (465 cusecs) at the Rocca d'Evandro power station and 18 0 cu. m. per sec. (635 cusecs) at the S. Ambrogio power station, which will give a total production of 131 million kWh per year.

Details of these two power stations are as follows:

FIRST POWER STATION (ROCCA D'EVANDRO)

Natural head	140·30 m. (460 ft.)
Installed power	2 × 22,000 kVA
Energy production per year	109,000,000 kWh
Delivery tunnel dimensions:	
Circular diameter ...	4·26 m. (14 ft.)
Area	13·86 sq. m. (149 sq. ft.)
Length	14·460 km. (9 miles)
Discharge tunnel:	
Length to the reservoir for the second power station is 5·042 km. (3·14 miles) of which 4·052 km. (2·52 miles) is tunnel and 0·990 km. (0·62 miles) is open channel.	

SECOND POWER STATION (SANT' AMBROGIO)

Head	20·10 m. (66 ft.)
Installed power	7,500 kVA
Energy production per year	22,000,000 kWh

The delivery head consists of the discharge channel of the first power station, which, in open air, has a length of 1,180 m. (3,870 ft.)

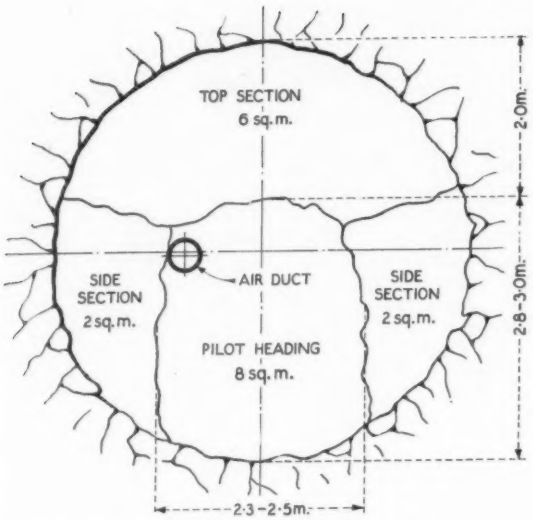
The planning of these power stations was undertaken by Prof. Carlo Drioli, General Director of S.M.E., Prof. Giacomo Baroncini, Director of the Plants, and Eng. Ugo Carotenuto, the Director of Works for the power station.

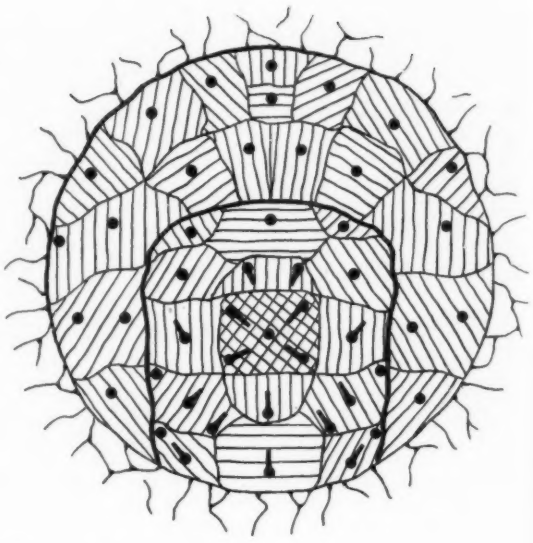
Fig. 3. The camp at Venafro, consisting of prefabricated houses dispersed on the mountainside

Rocca d'Evandro Delivery Tunnel

One of the most interesting aspects of the work for the Volturno - Garigliano power station is the delivery tunnel, which carries the waters from the Volturno, with a head of 140·30 m. (460 ft.) to the Rocca d'Evandro power station. This is essentially a pressure tunnel, of circular section, with an internal diameter of 4·26 m. (14 ft.) and having a length of 14·50 km. (9 miles). The first 5,500 m. (18,000 ft.) cross the plain of Venafro in a pipeline constructed above the plain and under the Volturno-Sulmona railway, the Rio S. Bartolomeo and some smaller roads.

For the succeeding 9 km. (5·6 miles) the tunnel goes through the limestone mountains which divide the Vol-





turno basin from that of the Garigliano. When passing the Casilina highway it is constructed as a reinforced-concrete bridge with five arches of 25 m. (80 ft.) width each.

The contracts for the construction of the tunnels were awarded by S.M.E. to Impresa Angelo Farsura of Milan with Silvio Arrigoni as director of works, and Impresa Nicola Rivelli of Naples with Nicchiarelli as director of works.

In this article we are concerned only with the work carried out by Impresa Farsura in the first part of the tunnel, which conveys the waters from the Volturno to the Garigliano. This work, which has now been completed, has attracted particular interest due to the exceptional results obtained. The tunnel driven by this company has the following data:

Total length (two adits, one with shaft)	5,436 m.	17,800 ft.
Diameter of tunnel:		
Excavation	4·95 m.	16 ft. 3in.
Finished	4·26 m.	14 ft.
Finished area	19 sq. m.	205 sq. ft.

Type of rock: Limestone of extremely variable character.

The work of excavation was started on February 1, 1950, from both ends at Venafro (Pozzo, Rocca Pipirozzi) and Mignano (Cannavinelle), driving in opposite directions on the same axis.

Tunnelling Operations

The tunnel was planned according to the full-face method. However, in order to ascertain the type of rock likely to be encountered, the excavation was

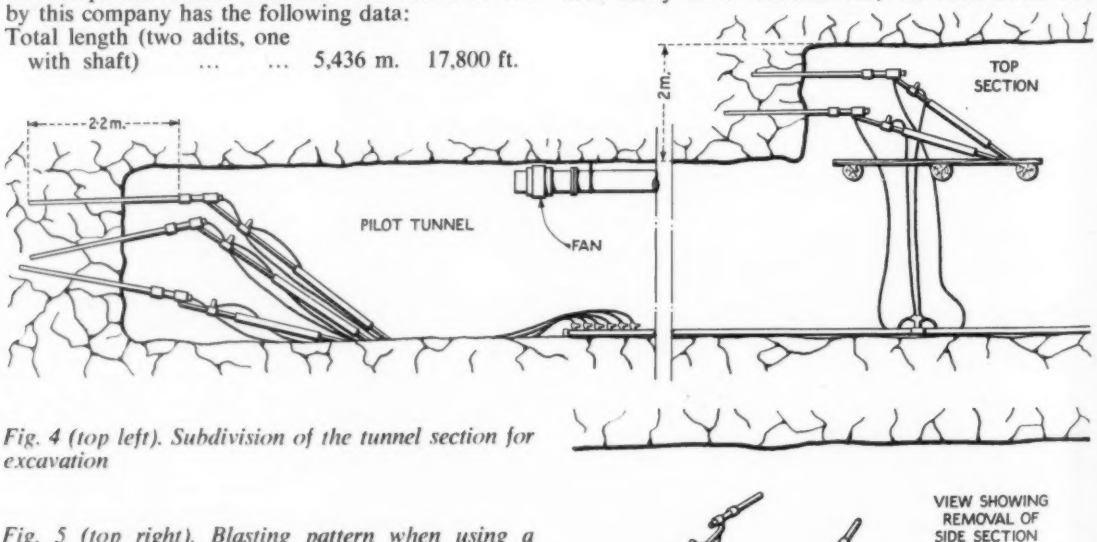


Fig. 4 (top left). Subdivision of the tunnel section for excavation

Fig. 5 (top right). Blasting pattern when using a bottom pilot heading

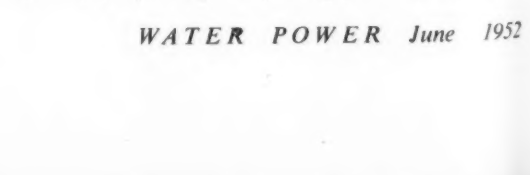


Fig. 6. Disposition of drills in the pilot and main headings

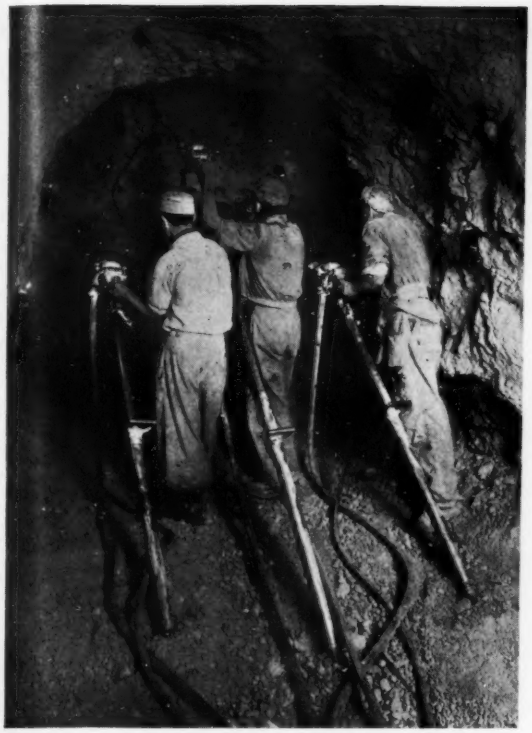


Fig. 7. Three Atlas RH 655W drills at work in the pilot tunnel

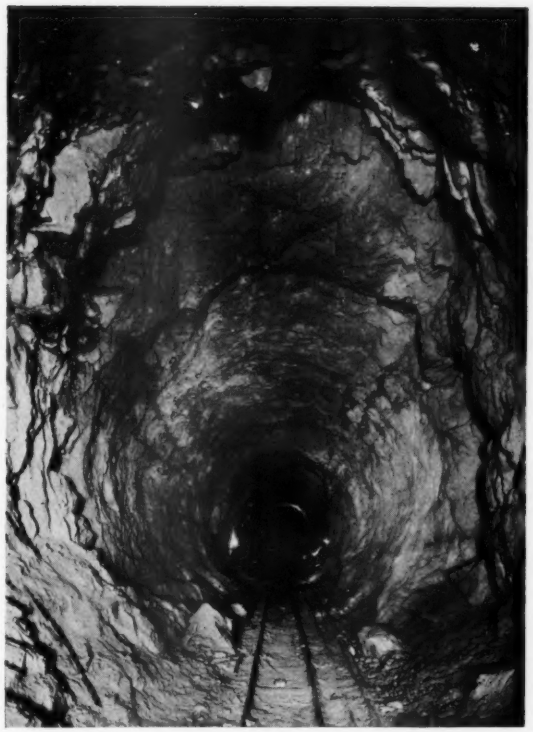


Fig. 8. A portion of the Rocca d'Evandro tunnel before being lined

carried out with a pilot tunnel which was kept at the bottom when the rock was hard and firm, but was carried along the top when the nature of the rock required immediate reinforcement. The full-face method was, therefore, generally followed by breaking down the top section in the first case (pilot tunnel at the bottom) and by breaking down the central section in the second case (pilot tunnel in the roof). The pilot tunnel was driven far in advance of the main heading, not in order to secure greater economy or to facilitate the work by dividing it into two sections (it would have been more economical, in fact, to have driven all on the same vertical section) but, on the contrary, mainly to allow a more exact and careful examination of the rock to be made and so adopt the best blasting methods. The pilot tunnel was, therefore, of an exploratory character (see Fig. 6). The excavation at the top and side sections kept pace of course with the pilot tunnel at a suitable distance in the rear.

Fig. 6 also shows the number and position of the rock drills used, all being of Atlas manufacture, viz:

3 type RH-655W, with pushers, in the pilot tunnel.
2 for the roof section.
2 for the side sections.

Seven rock drills were thus simultaneously at work in each heading and each drilled thirty 7-8 ft. deep holes in each shift.

Charging and Blasting

Drilling was conducted on the Swedish double-vee pattern, the firing scheme being shown in Fig. 4. Fig. 5 shows the firing sequence used where the rock was poor and less compact. The quantity of explosive used was 1·5 kg. per cu. m. (2·4 lb. per cu. yard) of rock with a composition of 1·1 kg. G.D.M. and 0·4 kg. Dinamon.

Ventilation

Ventilation of each heading was effected by a 35 h.p. electrically-driven fan for the main circulation of the air, assisted by a compressed-air-driven auxiliary fan for the quick dispersal of the gases after the blasts. This double system of ventilation allowed the workers to go back to work in a very short time—not more than about 8-10 minutes. The ventilation ducting was 400 mm. diameter (15¾ in.) and constructed of steel sheeting for the first 1,500 m. (5,000 ft.) after which canvas-rubber tubing was used for the remaining distance to the face.

Removal of the Spoil

The driving method used made mucking out an easy matter. In fact, as the pilot tunnel was driven far in advance of the top and side sections—about 60 m. (200 ft.)—it was possible for a whole train of 15 cars and a loader to be moved into the pilot tunnel before blasting the top and side sections of the main heading. Mucking out took place in the main tunnel and the pilot tunnel simultaneously, the operation being so timed that the whole train of cars and two loaders moved out together, making way for the two new drilling teams.

Organisation and Analysis of Working Time

For the construction of the part of the tunnel

assigned to the Impresa Farsura, 500 workers were employed. Of these, 60 were used from the approach adit of Magnano and 60 from the approach adit of Venafro. At each adit the workers were divided as follows:—

Pilot tunnel:

1	foreman
4	drillers
1	helper
1	loader driver
1	mechanic
8	for three shifts of 8 hours

Top and side sections:

1	foreman
6	drillers
2	helpers
1	loader driver
1	mechanic
11	for three shifts of 8 hours

When rock was encountered that made reinforcement and timbering necessary (Fig. 10), the drilling team was modified somewhat as follows:—

1	foreman
8	drillers
2	helpers
11	for two shifts of 10 hours

Normally the two approaching gangs worked in shifts of 8 hours with the excavation of two rounds per shift, viz. six rounds in 24 hours. As the advance in each round was about 1·80 m. (5·9 ft.), the average rate of advance in 24 hours was 10—12 m. (33—40 ft.). Consequently the advance from the two adits in 24 hours averaged 20—22 m. (66—72 ft.).

The division of the working time for each shift is given hereunder:—

Advance of loading machine ...	10	minutes
Mucking	90	„
Removal of loading machine ...	5	„
Advance of rock drills	10	„
Drilling	100	„
Charging and blasting	10	„
Ventilation	15	„
Each shift ...	240	minutes

Details of Typical Round in the Pilot Tunnel

Cars loaded (1,200 litres each) (42½ cu. ft.)	15
Number of holes drilled	20—30
Total length of holes drilled	70—75 m. (230—246 ft.)
Explosives used:	
GD1—M	20 kg. (44 lb.)
Dinamon	5 kg. (11 lb.)
Advance obtained	2—2·20 m. (6 ft. 6 in.—7 ft. 3 in.)

Use of Drilling Equipment

All the above details refer to the second part of the tunnelling work, carried out by the Impresa Farsura. The whole work may be divided into two periods,

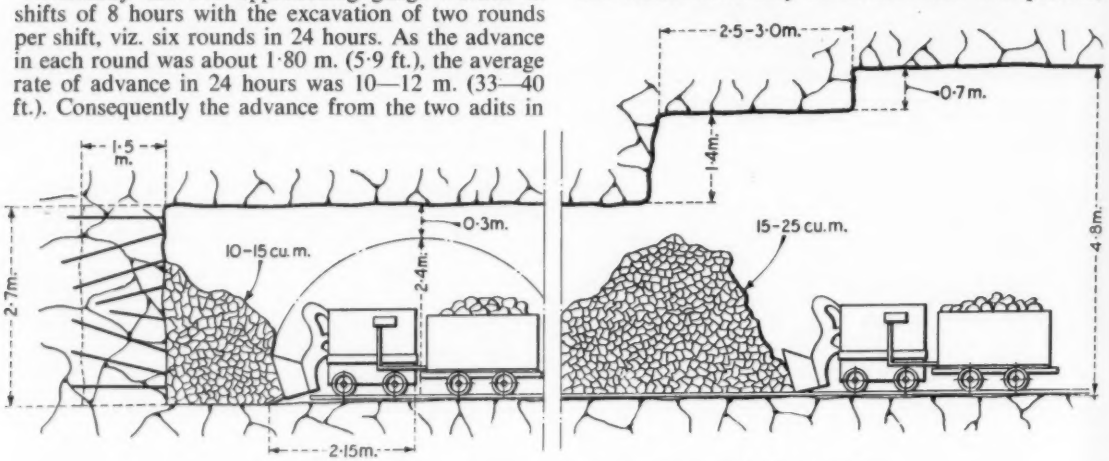


Fig. 9. Mucking out in normal rock using a bottom pilot heading

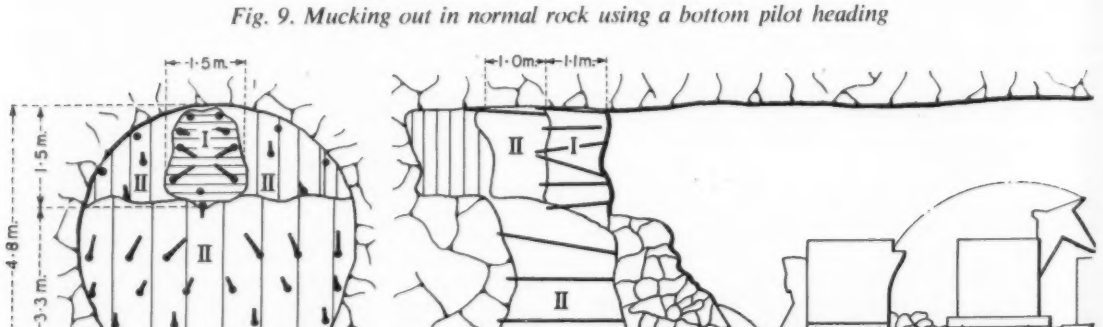


Fig. 10. Blasting pattern and mucking out in weak rock using a pilot heading at the top

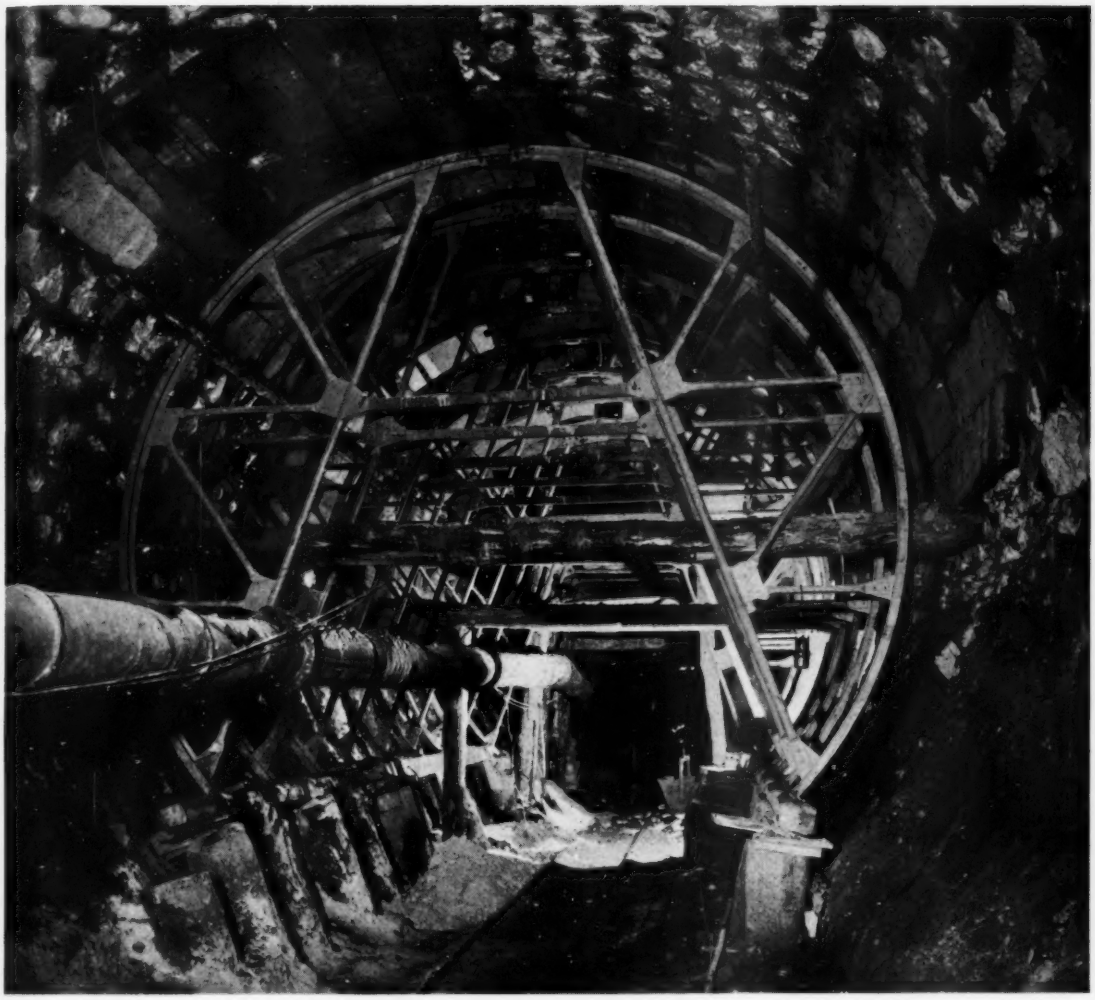


Fig. 11. Shuttering for placing the concrete lining

according to the drilling methods used; the first from February 1 to June 30 and the second from July 1 to December 10, 1950.

Excavation was begun on February 1 from the two adits, Venafro and Mignano, but owing to the necessity to obtain a more rapid rate of advance, the engineers of the Impresa Farsura discontinued the drilling methods and equipment previously used and changed over on July 1, 1950, to the Swedish method. This innovation resulted from a study that the engineers had made in the meantime of a number of tunnelling works in other countries, notably various hydro-electric schemes under construction by Electricité de France, where the Swedish method had been successfully applied for two years or more. So impressed were they with the results they saw that they decided to convert to the same method and equipment without delay. In France this method was based on the use of Atlas light rock drills, type RH-656W, used in conjunction with Sandvik Coromant tungsten-carbide-tipped drill steels and pneumatic pushers. An immediate improvement in the rate of advance resulted even before the miners had time to become conversant with the new technique, but as more experience was gained tunnelling records began to be obtained. One fact alone will be sufficient to indicate the success obtained. The completion of the work was timed for the end of May 1951, but against any original expectation, the work was finished on December, 10, 1950.

An interesting comparison of the footage obtained with and without the new equipment is as follows:—

Heading	Older Equipment	Swedish Method
	In 5 months' work (from Feb. 1 to June 30, 1950)	In 5 months and 10 days' work (from July 1 to December 10, 1950)
Mignano -	800 m. (2,620 ft.)	1,700 m. (5,580 ft.)
Venafro -	420 m. (1,380 ft.)	2,080 m. (6,820 ft.)
Total Advance	1,220 m. (4,000 ft.)	3,780 m. (12,400 ft.)

(Continued on page 236)

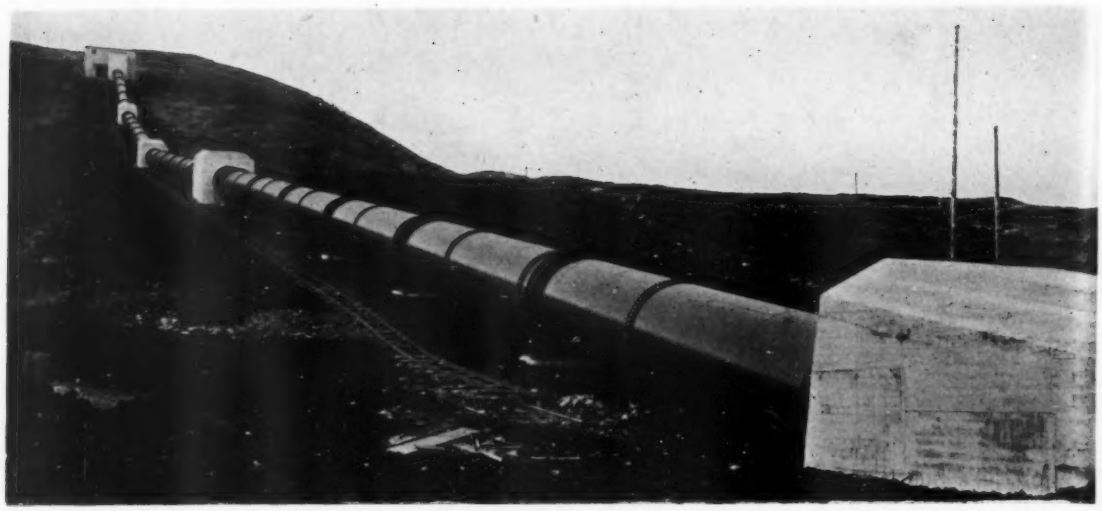


Fig. 1. The pipeline leading from the tunnel to Grudie Bridge power station

Testing Hydro-Electric Generating Plant

A comprehensive account of the official efficiency tests at the Grudie Bridge power station of The North of Scotland Hydro-Electric Board is contributed by The Harland Engineering Co. Ltd., Alloa, Scotland

ON October 10 1951 the official efficiency tests were carried out on one of the two 12,000 kW Harland Bruce-Peebles turbo-alternator sets at Grudie Bridge power station, which was described in a previous issue of this journal* and which has been running almost continuously since it was commissioned in December 1950.

As will be shown later, the overall efficiency of a hydro-electric generating set at any particular load is a function of net head and water consumption. Although no great difficulty or controversy is attached to the measurement of head or electrical output, opinions differ widely as to the best and most reliable method of measuring the discharge or rate of flow of water. B.S. 353 states that the best method depends upon circumstances, and recommends, if feasible, that more than one of the following recognised methods should be employed, viz.: current meters *(a)*, pitot tube *(c)*, pressure-time (Gibson) *(d)*, salt titration *(d)*, salt velocity *(d)*, travelling screen *(b)*, venturi meter *(c)*, volumetric *(b)*, and weir *(a)*.

The Swiss, on the other hand, go as far as to classify the above and four additional methods, into: *(a)* normal methods, *(b)* other methods suitable chiefly for laboratory measurements, *(c)* doubtful methods, and *(d)* methods not yet sufficiently tried out. The letters appended to the recognised methods listed above refer to this classification.

* WATER POWER, November 1951, page 424.

In this instance, it was agreed with the clients (The North of Scotland Hydro-Electric Board) and their engineers (Messrs. Merz & McLellan) that the salt velocity method, devised by the late Prof. Charles M. Allen of Worcester Polytechnic Institute, Worcester, Massachusetts, U.S.A., which has been successfully applied on a great number of hydro-electric installations in the United States and elsewhere (including Scotland) should be used, and The British Pitometer Co. Ltd., whose former managing director, Capt. E. R. Howland, holds the British rights for this method, were engaged to provide all the necessary instruments and equipment for the water-quantity measurements, and carry them out in the capacity of an impartial, independent authority.

This method of water measurement in closed conduits is based on the fact that salt in solution increases the electrical conductivity of water. Thus, if a quantity of salt solution is injected into the penstock upstream of a pair of electrodes, the time of passage of the brine may be recorded graphically on the chart of a recording ammeter equipped with one pen to record the injection and, at regular intervals, the timing signals from a precision clock, and another pen to record the increase in conductivity of the water produced by the passage of the brine past the electrodes. If the volume of the penstock between the brine injection point and a point midway between the electrodes has been accurately determined, the rate of flow may be calculated from the equation:

$$Q = V/t \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where Q is the discharge in cubic feet per second,
V is the volume in cubic feet, between the brine injection point and the point midway between the two electrodes,
t is the time in seconds, of passage of the brine between the brine injection point and the electrodes.

Measurement of Pipe Dimensions

As the pipe consists in the main of 23 ft. 9 in. lengths of fabricated tube, it was decided to number these lengths consecutively from the bottom, the length containing the electrodes being called No. 1. At length 31 the pipe is tapered to reduce the diameter from 7·700 ft. upstream to 7·330 ft. downstream.

The diameter of the pipe was measured by means of an expanding point gauge, calibrated in decimals of a foot. The divisions were such that it was possible to measure accurately to the nearest 0·002 ft. Two readings at right angles to each other were taken at the bottom (just clear of the rivets), centre and top of each length, i.e., six readings per length with approximately 11 ft. between each set of readings. The slip joints, just below each anchor block are flanged at each end, and no readings were taken at the slip joint itself—only at the flanged ends of the adjacent fabricated sections.

The diametrical measurements as far as the top end of length 44 were taken after the pipe had been painted. Subsequent measurements were made on unpainted pipe. A specimen plate which had been half-painted with the same number of coats as the inside of the pipe was lent by the civil engineers in charge of that section of the work, and it was found that the average total thickness of the four coats of paint was not more than 0·0065 in. The error, therefore, in neglecting the thickness of paint was no more than 0·028 per cent. on a diameter of 7·700 ft.

The readings of pipe diameters taken by the foregoing procedure were tabulated, and the positive or negative deviation from the nominal diameter calculated in each case. These deviations were then summated and the average diameters for the two sections of penstock on either side of the taper piece determined. These were found to be 7·3394 ft. and 7·7043 ft. respectively.

The distance between the midpoint of the electrodes and the injection point was measured as follows. The origin was taken as the point midway between the two electrodes, and measurement was made along the bottom of the pipe right to the end of length 53. The point of injection was determined by measuring along the top of the pipe to the centreline of the 4 in. gas hole from the end of length 53, and subtracting $2\frac{9}{16}$ in. from the length so determined. The length of the smaller-diameter section was found to be 638·250 ft. and of the larger-diameter section 466·386 ft.

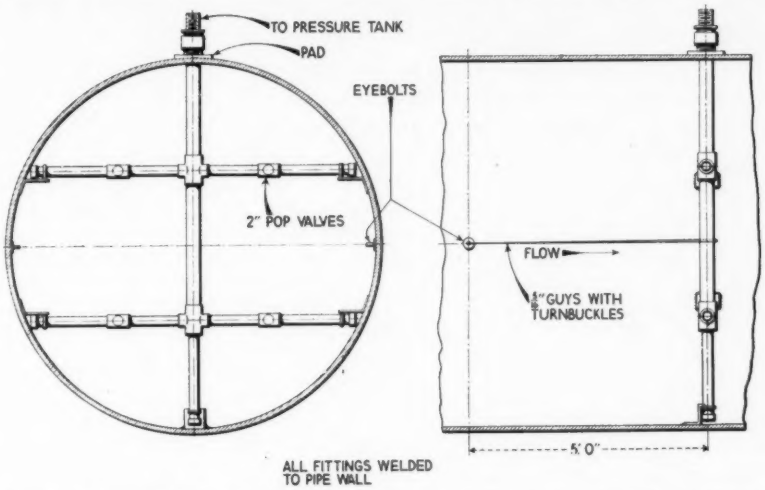


Fig. 3. Arrangement of brine pipes and nozzles at upstream end of penstock

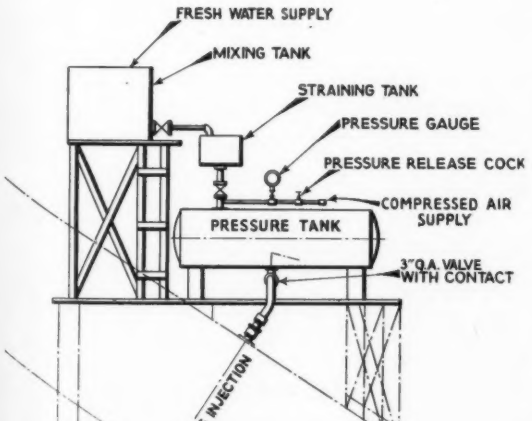


Fig. 2. Apparatus for injecting brine into penstock

The volume of the pipe was then calculated from the pipe lengths and average diameters thus determined, and found to be 48,745 cu. ft.

Determination of Time of Passage of Brine

For the determination of the time of passage of the brine t, a set of tanks and fittings, as shown in Fig. 2, was erected on a wooden platform at the brine injection point. A manhole and pressure tapping were provided on the penstock, the former to permit the assembly, inside the penstock, of the brine-injection pipes and pop-valves, shown in Fig. 3, and the latter to receive the pipe from the 3 in. quick-acting valves. A compressed-air cylinder (a total of 1,980 cu. ft. of compressed air was provided for the tests) was connected to the 1 in. socket shown in Fig.

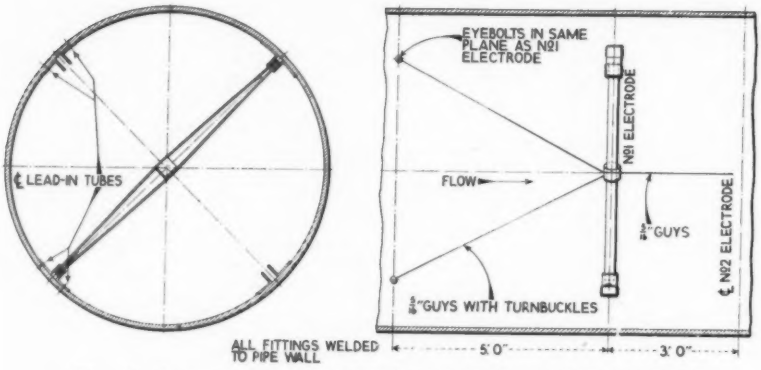


Fig. 4. Arrangement of electrodes at turbine end of penstock

2, and a 10 per cent. brine solution prepared in the mixing tank. 10 cwt. of undried vacuum salt was provided for this purpose. The injection pressure was adjusted to approximately 50 lb. per sq. in. above the pipeline pressure at that point.

At the electrode point a pair of electrodes was installed inside the penstock, as shown in Fig. 4, and connected up as shown in Fig. 5. The a.c. supply to the electrodes was taken from the 240 V station supply via a 200-440/110 V 2 kVA single-phase variable-tapping transformer.

A temporary shelter was erected in the vicinity of the electrodes for the recording ammeter, ballast board, switches and precision pendulum clock.

One of the recording-ammeter pens was connected in the electrode circuit and the other in series with a 6 V dry battery and with either the contacts on the precision clock or those on the quick-acting valve, depending on the position of the selector switch. The recorder chart was clockwork driven at a speed of 12 in. per min.

Following the installation of the equipment, the electrodes were "meggered" and the penstock refilled. The supply to the electrodes was switched on and the voltage varied by means of the variable tapping transformer until the required deflection of the recorder pen was obtained. A trial injection was then made to check the recording of the injection, electrode and timing signals, and thereafter the tests were proceeded with on the following lines.

The required load was applied to the generator, which was paralleled on to the grid, and as soon as load conditions were steady, the electrode point (which also acted as control station for the entire tests) instructed the injection point to inject in exactly one minute's time. A few seconds before the expiration of that minute, the chart of the recording ammeter was set in motion and the selector switch thrown over to the left (see Fig. 5) thus enabling the time pen of the recording ammeter to record the 2 sec. timing signals given by the contact on the precision pendulum clock. Just before the precise moment when the injection was due, the selector switch was thrown over to the right for the time pen to record the injection signal, on receipt of which it was returned to the left-hand position to enable the time pen to continue recording the timing signals. Upon arrival of the brine at the electrodes, the increase in conductivity of the water caused the electrode pen on the recording ammeter to deflect; due to the variation of water velocity over the pipeline diameter and also to partial dissolution of the brine in the course of its passage down the pipeline, this signal was not instantaneous but long drawn-out, thus describing a "cocked hat" curve on the recorder chart.

The electrode supply being a.c. (d.c. would cause electrolytic action on the electrodes), the deflection of the electrode pen is not a linear function of the current, thus necessitating the replotting of the curve to a linear scale. The time of passage of the brine t is then taken as the time interval represented by the distance between the centre of the rectangular diagram obtained from the 1 sec. brine injection and the centre of the area enclosed by the curve drawn by the electrode pen, redrawn to a linear scale. The centres of these areas were determined by cutting them out of heavy squared paper and balancing them on a knife edge.

It was decided to test the machine at six different loads and between three and six runs were taken at each load, with a total of 24 runs. At the higher loads, at which the rate of flow was relatively high, and the time of passage of the brine correspondingly short, runs were taken consecutively, but at the lower loads, at which the rate of flow was relatively low and the time of passage of the brine correspondingly long, considerable saving in time was achieved by overlapping the runs at each load (i.e. injecting a further shot of brine before the previous one had reached the electrodes). In view of the above procedure for determining the centre of area of the recorder diagrams, it was not possible to calculate the cusecs immediately, and therefore, in order to make certain that the readings were all of the right order, a rough check was made after one of the runs; from an approximate estimate of the time of passage t, in conjunction with the other relevant readings, the overall efficiency was calculated and was found to be of the right order.

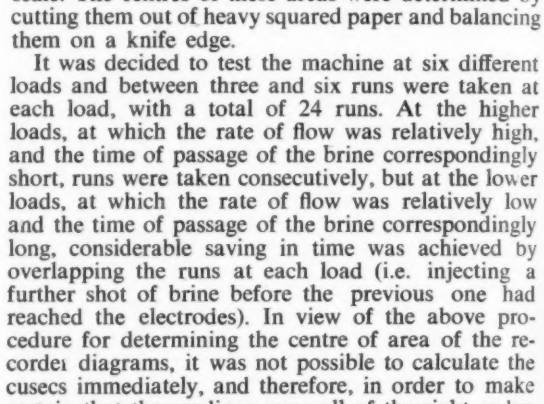


Fig. 5. Diagram of electrical circuits

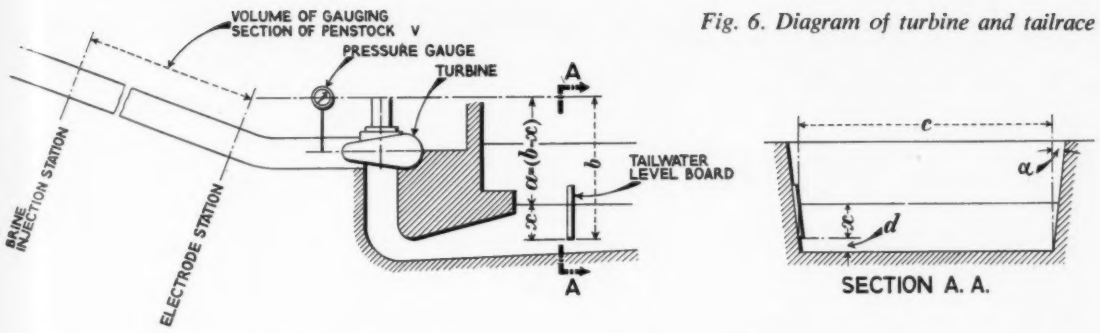


Fig. 6. Diagram of turbine and tailrace

The effective net head h is the sum of the pressure head P_e, the velocity head $\frac{v_e^2 - v_a^2}{2g}$ and the height of the centreline of the pressure gauge above tailwater level a.

where v_e is the velocity of the water at the pressure gauge tapping (feet per sec.).

v_a is the velocity of the water at the tailwater level gauge (feet per sec.).

g is the acceleration due to gravity (=32·2 ft. per sec. per sec.).

The pressure head was measured on a 10 in. standard pressure gauge, reading 0-1,000 ft. head of water. A special teepiece was inserted in the pressure pipe leading to the turbine-panel pressure gauge, in the vicinity of the tapping on the turbine spiral casing. A gauge cock was provided and adjusted to give the best possible stable reading consistent with accuracy of measurement. The standard pressure gauge was calibrated by deadweight tester in The Harland Engineering Co. Ltd.'s works immediately before and after the tests, and exactly the same readings were obtained in both cases. A calibration curve was plotted enabling the conversion of the pressure-gauge readings P to correct values of pressure head P_e.

The velocity of the water at the pressure gauge tapping v_e is given by:—

$$Q/\frac{\pi}{4}D^2 \quad \ldots\ldots (2)$$

where D is the pipe diameter at the pressure-gauge tapping (feet).

D was obtained from the drawing of the inlet taper pipe, on which the pressure-gauge tapping is situated.

The velocity of the water at the tailwater level gauge v_a is given by:—

$$Q/A \quad \ldots\ldots (3)$$

where A is the cross-sectional area of the tailrace at the tailwater level gauge (sq. ft.).

It is important that only that part of the tailrace width should be taken into account which corresponds to the unit on test, but since only one machine was running, the full width of the tailrace was taken in this case From the diagrammatic representation of the turbine installation and tailrace (Fig. 6), the cross-sectional area of a tailrace of trapezoidal section A is given by:—

$$x^2 \tan\alpha + x(2d \tan\alpha + c) + d(d \tan\alpha + c) \quad \ldots (4)$$

where x is the reading of the tailwater level gauge (feet).

c is the width of the bottom of the tailrace (feet).

d is the height of the bottom of the tailwater level gauge above the bottom of the tailrace (feet).

α is the slope of the tailrace walls.

In actual fact the cross-section of the tailrace in question was not a true trapezium, and an appropriate correction had to be made to equation (4) above.

It will also be seen that the height of the centreline of the pressure gauge above tailwater level a is given by:—

$$b - x.$$

where b is the height of the centreline of the pressure gauge above the bottom of the tailwater level board.

The tailwater level was read on the existing tailwater level gauge, which is situated on the left-hand bank of the tailrace (looking downstream), and calibrated in fifths of a foot; observations were taken from the right-hand side (looking downstream) of the draught tube and relief-valve gate hoisting gallery (tailrace platform). c and d were obtained from drawings, and the height of the centreline of the pressure gauge above turbine floor level was measured to enable b to be determined. Thus it will be seen that the only variables in the determination of h are P and x. Readings of these two quantities were taken at half-minute intervals from receipt of the "start" to the "stop" signal from the test control point (electrode station), and the mean of each set of readings was used for the efficiency calculations.

Tests were carried out at 113·4, 100, 83·33, 75, 66·67 and 50 per cent. full load, and the exact generator output W_G at these nominal loads was taken as the mean of half-minute readings of the wattmeters for each run, using the two-wattmeter method.

With the exception of the copper losses, which were calculated from the currents and winding temperatures actually obtained during the tests, the generator losses were determined by the generator manufacturers and main plant contractors, Bruce Peebles & Co. Ltd., from works tests, and the generator efficiency η_G calculated by the "summation of losses" method. This enables the turbine output W_T and turbine efficiency η_T to be calculated.

B.S. 353, the Swiss Rules for Hydraulic Turbines, and the Test Code for Hydraulic Prime Movers published by the American Society of Mechanical Engineers, all lay down different rules as to whether or not such auxiliaries as bearing-oil pumps, governor-oil pumps, cooling-water pumps, etc., should be charged to the turbine, so that it is necessary to specify the basis on which the efficiencies are calculated. In this instance, it was agreed that all measurements of

output should be made as near to the alternator terminals as practicable and that the power required for auxiliaries, except for excitation of the alternator, is to be supplied independently of the turbine under test. Since on this installation, with the exception of the permanent-magnet generator (which has an output of less than 1 kW and was therefore negelected) the auxiliaries are all electrically driven, and since, as mentioned above, all measurements of electrical output were in fact made at the generator terminals, the power required for the auxiliaries formed part of W_G and was therefore not charged against the unit.

Owing to adverse line conditions the specified voltage and power factor could not be obtained in every case, so that the best possible combination of voltage and power factor had to be contended with. Once set, the load setting was not changed throughout each run (or set of runs), and as a precaution the load-limiting device on the governor was set to the appropriate gate opening.

Successful intercommunication was achieved by interconnecting six field telephones, located at the following points: (1) brine injection point, (2) generator floor (electrical instruments), (3) control room (load adjustment), (4) electrode station, (5) turbine floor (pressure gauge), and (6) tailrace platform (tailwater level gauge).

The electrode station, which, as mentioned previously, acted as control point for the entire tests, was thus able to call up all stations simultaneously, and having ascertained that they were all "netted in," instructed them to stand by for the "start" signal. The brine injection point was then instructed to proceed with the injection, and receipt of the injection signal on the recording ammeter was immediately relayed over the telephone to all stations, who, while still listening in, proceeded to take their readings until receipt of the "stop" signal from the electrode station on completion of the run. The distances between the various test points, the number of quantities to be measured, the exact timing and synchronisation required, and the limited time available, made first-class intercommunication essential. Only one of the 24 test runs had to be repeated owing to failure of telephone communications, and on the whole the system adopted proved very successful in that it helped to keep delays between runs to an absolute minimum.

Calculations

The overall efficiency of the turbo-alternator set is given by:—

$$\eta_o = \frac{\text{generator output (kW)}}{\text{turbine input (H.P.)} \times 0{\cdot}746} \times 100\%$$

The input to the turbine is given by:—

$$\frac{Qwh}{550} \text{ H.P.}$$

where w is the weight per cubic foot of water ($=62{\cdot}4$ lb. per cu. ft.).

Therefore

$$\eta_o = \frac{100 \times 550\, W_G}{0{\cdot}746 \times 62{\cdot}4\, Qh}\ \% = \frac{1181\, W_G}{Qh}\ \%$$

But $Q = V/t$

and $h = a + P_e + \dfrac{v_e^2 - v_a^2}{2g}$ ft. of water.

whence from equations (1), (2), (3) and (4),

$$v_e = V/\frac{\pi}{4}D^2 t \quad \text{and}$$

$$v_a = \frac{V/t}{x^2 \tan\alpha + x(2d \tan\alpha + c) + d(d \tan\alpha + c)}$$

Also, from Fig. 5:—

$$a = b - x$$

and therefore, expressing the overall efficiency in terms of the known constants and measurable variables, we get:—

$$\eta_o = \frac{1181\, W_G t}{V\left\{6 - x + P_e + \dfrac{V^2}{64{\cdot}4\, t^2}\left[\dfrac{1{\cdot}62}{D^4} - \dfrac{1}{[x^2 \tan\alpha + x(2d \tan\alpha + c) + d(d \tan\alpha + c)]^2}\right]\right\}}$$

The turbine efficiency is given by:—

$$\eta_T = \frac{\eta_o}{\eta_G} \times 100\%$$

and may be calculated from the above and a knowledge of the generator efficiency η_G.

If, as was the case in this instance, the efficiency guarantee takes the form of a weighted efficiency, computed from three or more different loads, the efficiencies at the specified loads are taken from a curve plotted from the test results.

Correction for Head

At the time of the tests, the specified net head for which the guaranteed efficiencies applied was not available, and the unit had to be tested on a net head in excess of the agreed permissible 3 per cent. tolerance limit. If the speed of the unit can be varied, then it is possible to compensate for this discrepancy by varying the speed in the ratio of the square roots of the heads. In this case, however, the generator was synchronised on to the Highland grid for loading purposes, which made it impossible to vary its speed. An attempt was made, however, to analyse the test results in such a way that such valid deductions could be made from the results available as are required to prove that the efficiency guarantees given had, in fact, been met.

This was done in two different ways.

Method 1 consisted of drawing a curve of turbine efficiencies against turbine output, multiplied by the ratio of the rated net head to the test head, to the power 3/2. It may, however, only be considered a partial correction since it takes account of differences in head only and not of the effect of change in unit speed on efficiency.

It so happened that expected efficiency curves were originally submitted for three different heads and that one of these heads was approximately the same as the mean head prevailing at the time of the tests. Method 2 therefore consisted of comparing the weighted test efficiency with that derived from this efficiency curve.

A field test such as this requires a considerable amount of preparatory work and full co-operation

(Continued on page 236)

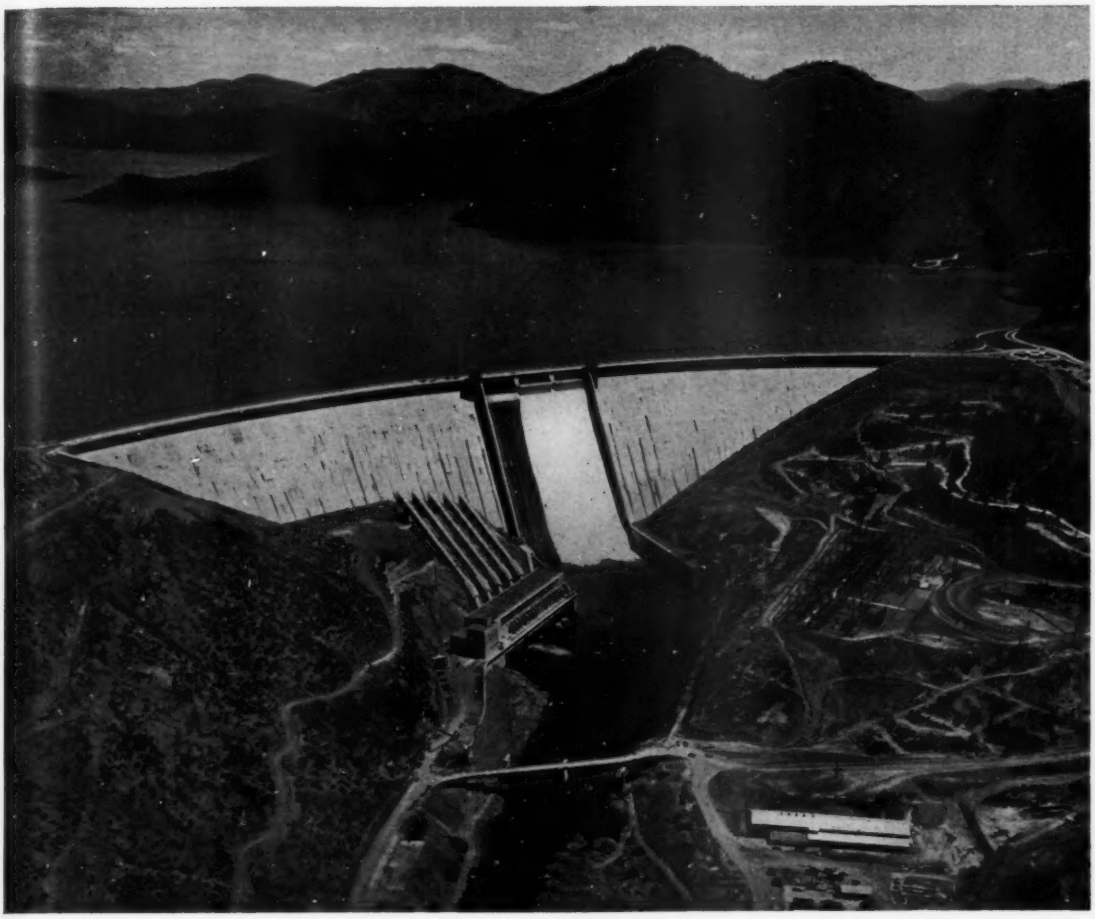


Fig. 1. Shasta Dam, the second highest in the world, and power station

The Central Valley Project—I

Stephen H. Poe, of the United States Bureau of Reclamation, gives an outline of this huge multiple-purpose project. The Central Valley embraces 18,000 square miles of the State of California and is nearly 500 miles long. The engineering works include the second highest dam (Shasta) and the second largest pumping plant (Tracy) in the world, one of the world's largest dams at Friant, and two immense canals (Delta-Mendota and Friant-Kern). A summary is given of work now in progress

IN August 1951, the first integrated operation of the Central Valley Project in California was begun, bringing to a climax 14 years of construction on the project by the United States Bureau of Reclamation. The event was of great significance because it marked the beginning of the unification of the water resources of the valley and the opening of a new era in the development of the rich Central Valley agricultural empire, bringing a new wealth to the district.

From an engineering standpoint, the Central Valley Project exemplifies the Bureau of Reclamation's progress in the design and construction of huge multiple-purpose developments. Features of the project are Shasta dam, world's second highest; Friant dam, one of the world's largest concrete barriers; Tracy pumping plant, second largest in the world; and the huge man-

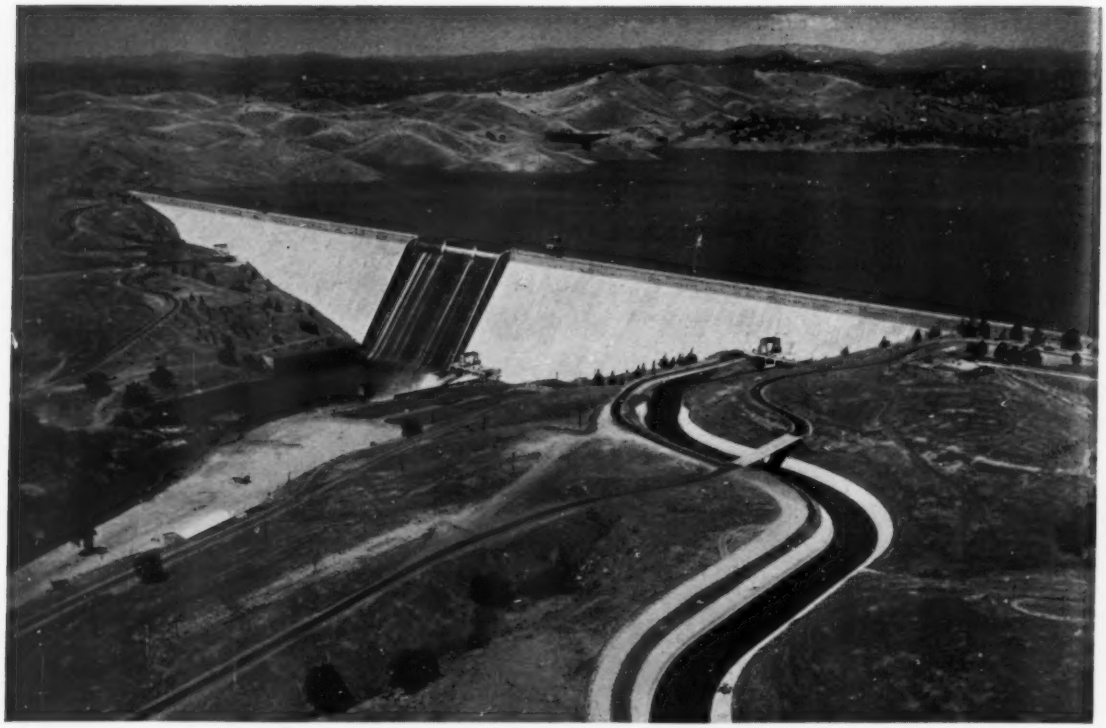


Fig. 2. Friant Dam, showing the commencement of the Friant-Kern canal on the right

made rivers, the Delta-Mendota and Friant-Kern canals. These and the assemblage of associated dams, canals, pumping plants, power plants, transmission lines, and irrigation distribution systems are integrated into a unified plan of service to bring new and supplemental water to vast reaches of the far-flung project.

The following series of notes describes the history of the Central Valley development, purpose of the project, and the project's major features.

History and Purpose of the Project

Interest in the water problems of the Central Valley began with the earliest history of California as a state, when in 1850 the first legislature passed a law requiring the preparation of plans for improving navigation, providing drainage, and furnishing irrigation water. During the succeeding 70 years many irrigation, flood-control, and hydro-electric projects were constructed. In 1919, the Chief Hydrographer of the United States Geological Survey submitted to the Governor of California a plan for the co-ordinated development of the water resources of the Central Valley. This created state-wide interest, and in 1921 the state legislature made the first of a series of appropriations for investigations of plans for the conservation, control, storage, distribution, and application of all the waters of the state. Intensive investigations by the state continued for 10 years until 1931 when the Division of Water Resources submitted to the legislature the State Water Plan, a comprehensive plan for utilising the water resources of the Central Valley.

In the meantime, serious water problems were developing particularly in the upper San Joaquin Valley, where the overdraft on ground water caused a continued lowering of the water table, and in the Sacramento —San Joaquin Delta where the encroachment of saline water endangered the industrial and agricultural development of the Delta region. In 1933, the state legislature approved the Central Valley Project Act which provided for the construction, operation, and maintenance of a system of works designated as the Central Valley Project, comprising essentially Shasta dam and reservoir, Contra-Costa canal, Delta cross channel, Delta-Mendota canal, Friant dam and reservoir, Madera canal, Friant-Kern canal, facilities for generation and transmission of electric energy, and such other units as may be added from time to time. This Act was also approved by a referendum of the people in 1933.

Efforts toward obtaining financial assistance on the initial units of the Central Valley Project, begun as early as 1929, were continued until 1935 when the project was authorised as an undertaking of the United States Bureau of Reclamation. Construction on the project began in 1937.

The Central Valley Project serves a large part of the central section of the State of California. The Central Valley itself, embracing 18,000 square miles, is nearly 500 miles long, extending from the foot of mount Shasta to the Tehachapi range, about 50 miles wide between the foothills of the Sierra Nevada and the coast range and varies in elevation from sea level to 400 ft. It is a huge platter-like valley, its high rim broken only at San Francisco bay through which its two principal rivers, the Sacramento and the San Joaquin, after joining in a common delta, flow out to the Pacific Ocean.

The Sacramento river has its source near mount

Fig. 3. Keswick Dam and power station, nine miles downstream from Shasta

Shasta at the northern end of the basin and flows southerly through the entire length of the Sacramento Valley. The San Joaquin river drains the northern half of the San Joaquin Valley. The river rises at the crest of the Sierra Nevada north-east of Fresno, flows down the western slope until it reaches the main valley, then turns north-westerly down the trough of the San Joaquin Valley.

The Central Valley includes 10 million acres of irrigable land surrounded by a mountain watershed of 40,000 square miles. The valley has been endowed with a rich soil and the elements of plant growth and temperature conducive to sub-tropical fruit culture all the year round. It is famous for its raisins, table grapes, and sweet wines. Major crops include peaches, figs, apricots, nuts, plums, olives, oranges, melons, alfalfa, cotton, grain, asparagus and potatoes.

The water problem of the Central Valley is mainly one of conservation. There is abundant water, but the agricultural development in the San Joaquin Valley in the last quarter century has far outstripped the natural limitations of the unregulated water supply; the water resources are out of balance with the irrigable land. Geographically, the Sacramento Valley in the north has tributary watersheds producing two-thirds of the water and has agricultural land with only one-third of the irrigation need, whereas the San Joaquin Valley in the south has one-third of the water and crop lands with two-thirds of the need. Seasonally, there are periodic floods and droughts—almost all the rain and snow falls in the few months of winter and early spring and two-thirds of the combined waters of the two rivers escape to the ocean during the non-irrigation season, leaving a shortage in the late summer and fall when water is most needed.

The Central Valley Project effects both the geographical and the seasonal redistribution of the water of this great valley to correct a series of maldistribution problems. By the conservation and regulation of the water resources of the valley about 1,000,000 acres of highly cultivated agricultural land is to be furnished with a supplemental supply of water; navigation is improved; salt-water intrusion in the Delta area is repelled; water is provided for domestic and industrial uses; fish and wildlife are conserved; recreational facilities are provided; and electric power is generated.

Shasta Dam and Power Plant

Shasta dam, the project's largest structure, is located on the Sacramento river 12 miles north of Redding, California, and five miles below what was formerly the confluence of the Pit and Sacramento rivers. Shasta dam now stores the spring run-off of the Sacramento river, releasing it as needed throughout the year for irrigation in the Sacramento Valley and for transfer as supplemental water to the San Joaquin Valley. In addition, Shasta dam develops power for use in the comprehensive Central Valley Project scheme.

Shasta dam is the third largest concrete dam in the world. The dam is a curved gravity structure 602 ft. from the lowest foundation to the crest and 3,500 ft. long at the crest.

In designing the dam its height was fixed within fairly close limits by the storage requirements of the project as a whole which established a gross reservoir storage capacity of 4½ million acre-feet on the Sacramento river. Of this capacity, the top 500,000 acre-feet is used primarily for flood control, and a dead storage volume of 500,000 acre-feet is maintained to provide a minimum power head of 238 ft. At the left

end of the dam, weathering affected the competency of the rock to great depth. To obviate removal of great quantities of material unsuited for foundation of a concrete dam, an earth- and rock-fill dam, 525 ft. long and 115 ft. high, was constructed in this area.

Shasta spillway, 375 ft. long, was designed for a maximum capacity of 185,000 cusecs at the full water reservoir surface elevation. Under maximum discharge conditions, the water drops 480 ft. on the downstream face of the dam, creating a dynamic energy of 7,200,000 h.p. A sloping apron at the base of the spillway provides for dissipation of this excessive energy.

The flow over the crest of the spillway and the water surface elevation of the reservoir above the elevation of the spillway crest are controlled by three floating-type drum gates. These gates are each 110 ft. long and have an effective operating height of 28 ft. They are automatically operated so that the discharge can be controlled closely within given limits.

When outflow from the reservoir is required in amounts greater than the discharge from the power-house turbines, the excess is discharged by the outlets, 18 conduits arranged in three tiers. All are 102 in. diameter welded steel pipe passing horizontally through the dam. Within a very few feet of the downstream face they turn downward abruptly until nearly parallel with the face so that the issuing jet will merge with minimum interference with the overflowing sheets from the spillway when it is in operation. The total combined capacity of the conduits is 65,000 cusecs.

Construction on Shasta dam began in September 1938. By the end of 1939, the greater part of the excavation had been completed. During the first 10 months of excavation, material was removed from the foundation of the dam at the average rate of nearly 230,000 cubic yards per month; the record month was 476,000 cubic yards. The first concrete was placed in the dam on July 8, 1940, and the first diversion of the river took place in August 1940.

By the end of 1940, 4,150,000 cubic yards of material had been excavated for the dam in the power plant, and 500,000 cubic yards of concrete had been placed. Concreting proceeded at an average rate of around 2,000,000 cubic yards a year until the six-millionth yard of concrete had been placed in December 1943. The last bucket of concrete was placed on the spillway bridge of the dam in December 1944, marking the production and placement of 6,535,000 cubic yards of concrete in the dam and appurtenant works.

The last of the work required under the major contracts for construction of Shasta dam was completed on June 20, 1945, on which date the United States accepted the Shasta dam and power plant. At the beginning of 1944, there was storage of only 18,000 acre-feet at Shasta reservoir. One year later, the reservoir contained a million and a half acre-feet. By December 1945, the storage elevation had increased to three and a quarter million acre-feet. The total capacity of Shasta reservoir is 4,493,000 acre-feet.

Shasta power plant, on the right bank of the river downstream of the dam, is a huge structure, although dwarfed by the immensity of the dam. In the power-house are five 103,000 h.p. turbines driving five 35,000 kVA generators, and two 4,250 h.p. turbines driving two 3,000 kW station-service generators. Five 15 ft. inside-diameter welded steel penstocks pass through the dam and convey water from the reservoir to the power plant. Water for each of the two station-service units is tapped from two of the main penstocks. The first power from the power plant was placed on the line on July 14, 1944. The last generator was placed in service on April 27, 1949.

Cooling pipes were installed throughout Shasta dam on top of each 5 ft. lift of concrete as it was placed, and at an average horizontal spacing of 5½ ft. to provide a means of minimising the temperature rise associated with the heat of hydration of the cement in the concrete. River water for preliminary cooling was circulated through the pipes from pipe headers located on the downstream side of the dam or in galleries in the spillway section. Final cooling to specified temperatures was by water refrigerated in a special plant on the left abutment.

To control the development of cracks that would normally be associated with such temperature changes, a system of contraction joints at approximately 50 ft. intervals both parallel and perpendicular to the axis of the dam was incorporated in the dam. Thus, the dam was broken into a staggered series of columns roughly 50 ft. square. A system of piping and grout outlets was provided, and after completion of cooling and attainment of final volume of the dam, the contraction joints were filled with cement grout.

Grout pumped into a series of vertical holes drilled into the foundation near the upstream face of the dam forms a continuous curtain or barrier against the infiltration of water under the dam. Development of pressure from any slow seepage of water is inhibited by a system of vertical relief drains immediately downstream from the grout curtain. Another system of 5 in. porous-tile drains of the concrete near the upstream face of the dam serves to prevent the development of internal water pressure in the mass of concrete itself.

A feature of the construction of Shasta dam was the immense cableway head tower projecting 466 ft. above ground and anchored over 100 ft. deep into the rock of a small promontory immediately upstream from the dam on the right abutment. This single-cantilever structure was designed to take the surging loads from seven cableways radiating outward from the top of the tower. All cableways had movable tail towers, three of them being as much as half a mile away from the head tower. The carriages for all cableways ran on a 3 in. cable weighing about 22 lb. per running foot. This cable way system handled over 19,000,000 tons of material during the construction of the dam.

During the construction of Shasta dam, instruments were embedded and observational points were established from which to obtain data and measurements for checking the assumptions used in the design of the dam and also to record its structural behaviour. A seismograph station was set up about three miles from the dam to record normal seismic behaviour of the region and to detect any additional seismic activity due to the increased load upon the earth caused by the reservoir. (The weight of the reservoir is about 6¼ billion tons.)

Friant Dam

Friant dam is located on the upper San Joaquin river about 20 miles north of Fresno and 21 miles east of Madera, California. The dam is a straight gravity-type structure, 319 ft. high and 3,488 ft. long at the crest and contains 2,135,000 cubic yards of concrete. The dam serves the dual purpose of storage for irrigation

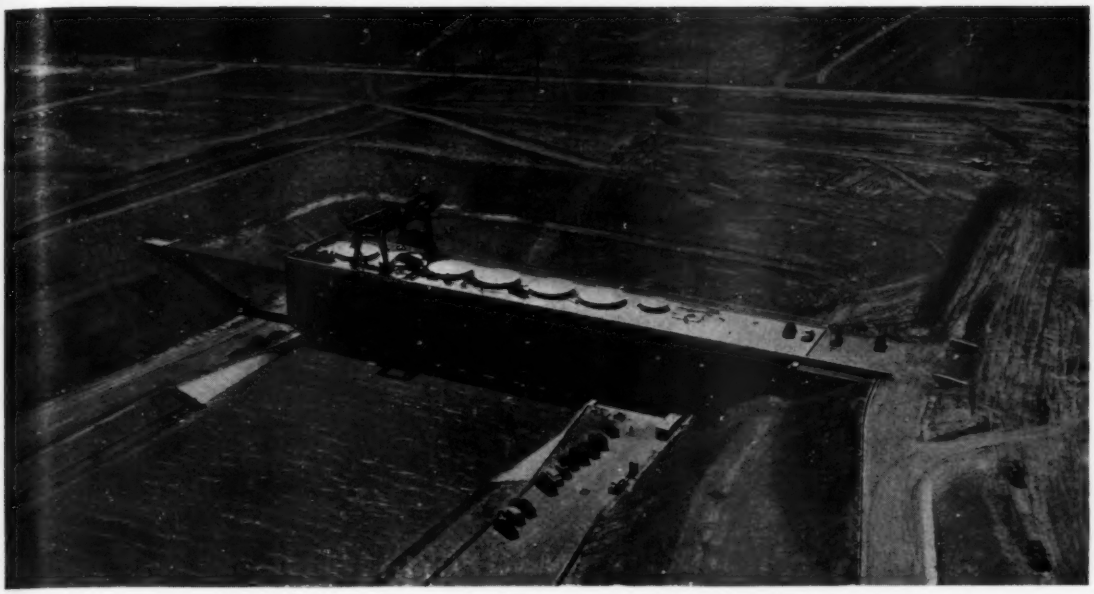


Fig. 4. Tracy pumping plant — the second largest in the world

and flood control. Water stored by the dam is distributed by a gravity system through two main arteries, Friant-Kern and Madera canals.

Millerton lake, the reservoir impounded by Friant dam, is 15 miles long and has a gross storage capacity of 521,000 acre-feet. The live storage above the canal headworks, exclusive of that reserved for flood control, is 360,000 acre-feet. The top 70,000 acre-feet in the reservoir is reserved for flood control. Actually the entire live-storage capacity of 430,000 acre-feet in conjunction with the canal diversion operates effectively for flood-control purposes. The dead storage of approximately 90,000 acre-feet forms a permanent lake about 10 miles long, providing excellent possibilities for boating, fishing, and other recreational uses.

Three 100 by 18 ft. drum gates control the overflow spillway which provides for a capacity of 92,000 cusecs. Four 110 in. conduit outlets located just adjacent to the spillway section are controlled by two 105 in. needle valves and two 102 in. tube valves hydraulically operated. In addition to these outlets, there are four 96 in. and two 78 in. valve-controlled outlets for the Friant-Kern and Madera canals. These outlets discharge into stilling basins and have a combined maximum capacity of 16,000 second-feet. On the upstream face of the dam, steel coaster gates operated by a travelling gantry crane on top of the structure provide for emergency closing of the canal outlets.

The contract for construction of Friant dam was awarded on October 9, 1939, and work at the site was begun by the contractor on November 3, 1939. The first bucket of concrete was placed in the dam on July 29, 1940; the millionth cubic yard was placed on May 31, 1941. The maximum concrete yardage placed in any one day amounted to 9,000 cubic yards and the maximum monthly total occurred in August 1941, when 228,000 cubic yards were placed. Friant dam was essentially completed on February 12, 1943. Further construction of the dam was halted by World War II. Installation of the drum gates and control equipment was completed in October 1947.

Most of the mass concrete in the dam and appurtenant structures was placed from a steel trestle, 2,200 ft. long, parallel to and 129 ft. downstream from the axis of the dam. From this trestle two hammerhead cranes and two whirley cranes placed the concrete. The trestle was embedded in the concrete as the structure rose.

Transverse contraction joints divide the dam into blocks 50 ft. wide in the abutment section and 56 ft. wide in the centre or storage section. As longitudinal joints were not provided, the individual blocks varied in length from about 30 ft. at the extreme abutment to a maximum of 270 ft. in the central section.

An interesting feature of Friant dam construction was the use of pumicite in the mass concrete for the dam. Specifications called for the substitution of pumicite by weight for 20 per cent. of the specified amount of cement. The large deposit of pumicite, a naturally occurring pozzolan, in the immediate vicinity of the dam site, led to consideration of its use as an ingredient in the concrete. It was found that the addition of an appropriate amount of pumicite made it possible to use mixes leaner than the usual fill of cement per cubic yard. At the same time, this practice was found to effect a considerable reduction in the heat of hydration or setting of the cement and made a more workable mixture and a denser, more nearly watertight concrete.

A supplemental contract permitted the contractor to recover the gold found in the gravel for the Friant dam concrete. Tests were made of several methods of recovering the gold, and it was finally determined that screen-lined chutes would be most efficient. At the end of the first month of construction more than $8,000 worth of gold was obtained.

Keswick Dam

Construction of Keswick dam began in October 1941, and was completed in October 1950, following the end of World War II. The dam, located about 10

miles downstream on the Sacramento river from Shasta dam, is designed to provide an afterbay regulating reservoir for Shasta dam. Keswick re-regulates releases from Shasta reservoir so that peak power production for short periods at the Shasta power plant will not create a hazard or interfere with other uses on lower reaches of the Sacramento river.

Keswick dam is a concrete gravity structure 159 ft. high above foundation, 1,046 ft. long at the crest, and has a volume of 197,000 cubic yards.

Keswick power plant, built as a part of the dam, contains three main generating units, each with a capacity of 25,000 kW driven by 34,600 h.p. hydraulic turbines. The output of these generators augments the power developed at Shasta and feeds directly into the service lines from Shasta's power-house.

The power-house structure measures 270 ft. along the axis of the dam. Water is admitted to each of the three turbines in the structure through three penstock passages, each 12 ft. 6 in. wide, leading to a scroll case formed in the power-house structure.

The spillway at Keswick dam is an ogee section controlled by four regulating gates each 50 by 50 ft. The maximum discharge capacity of the spillway is 250,000 cusecs. The outlet works consist of one outlet pipe 60 in. in diameter for the dam's fishtrap and three unlined penstocks controlled by three fixed-wheel gates each 12 ft. 7 in. by 23 ft. 5 in.

Keswick dam also protects the heavy fish run in the Sacramento river. The central section of the dam was designed as a fishtrap from which the salmon are removed and hauled approximately 40 miles in tank trucks to a hatchery on Battle Creek, a tributary of the Sacramento river. Here the eggs are removed, fertilised, and hatched, and released.

The fishtrap section is adjacent to the spillway and measures 64 ft. axially and 274 ft. along the direction of flow. It contains four holding pools, each 20 by 30 ft. 6 in.

Concrete was placed in Keswick dam in 5 ft. lifts, each surface being thoroughly cleaned by sandblasting before new concrete was applied. The excessive heat of the Sacramento Valley summer, which may reach 120°, brought special problems in the construction of the dam since specifications required that the concrete should be placed at a temperature not exceeding 85°. Spray nozzles discharging river water were used in the aggregate to precool the materials, and placed concrete was sheltered from the direct rays of the sun for three days and kept moist for 14 days while curing. Water from the river, without artificial cooling, was used for this purpose and also for circulation to the cooling pipes embedded in the concrete.

Tracy Pumping Plant

Tracy pumping plant is a major unit of the Central Valley Project. The plant has been referred to as the "heartbeat" of the project because it is the prime mover in the transfer of surplus water from the Sacramento river to the San Joaquin Valley. The plant discharges a maximum of 4,600 cusecs from an average elevation of 4 ft. above sea level to an average elevation of 201 ft. above sea level. The installation consists of six vertical pumping units each rated at 767 cusecs at a head of 197 ft. The Tracy pumping plant is the second largest pumping installation on any Bureau of Reclamation project, exceeded in size only by the Grand Coulee pumping plant of the Bureau's Columbia Basin project in the State of Washington.

The pumping plant, located about 10 miles east of Tracy, California, was constructed on the Delta-Mendota canal, approximately 13,400 ft. above its intake on the Old River—the old course of the San Joaquin river. Each pump discharges through a 108 in. diameter butterfly valve into a 10 ft. diameter steel discharge pipe. These pipes are paired into three 15 ft. diameter discharge pipes approximately one mile long. They discharge through a syphon structure into the Delta-Mendota canal.

It is anticipated that when the irrigation facilities are fully developed for the Central Valley Project and in operation, the maximum demand for water will occur in July and will be approximately 4,250 cusecs. Minimum demand is expected to occur in December and to be about 500 cusecs; the yearly average demand is estimated at about 2,060 cusecs. To meet this varying demand, it is expected that all six pumps will be operated in July and part of August.

The overall length of the plant is 362 ft. and its overall width is 96 ft. It is a reinforced-concrete structure of the semi-outdoor type. The plant is divided roughly into two parts, one consisting of a six-main pump bay having a length of 243 ft. and a service bay having a length of 119 ft. The roof deck supports a 100 ton gantry crane for lifting the plant's equipment.

The plant is supported on timber piles. The foundation is at an elevation of approximately 30 ft. below sea level. The centre line of the scroll-case of the pumps and of the butterfly valves in the discharge lines is at elevation 0 (sea level).

The Tracy pumps are of the vertical-shaft single-impeller single-suction centrifugal type. They are designated as 84 in. vertical volute pumps of the closed-type impeller design. The pumps are designed to have stable operation under all heads within a range of 174 ft. to 204 ft., the maximum input to the pumps not to exceed 22,500 h.p. over the entire operating range.

Each of the six main motors in the plant is rated at 22,500 h.p., 13,600 V, 0·95 leading power factor, three phase, 60 cycle, 180 r.p.m. The motors are of the synchronous vertical-shaft type with the thrust bearing and upper guide bearing located above the rotor and a lower guide bearing below the rotor. The motors are totally enclosed and are air cooled with forced surface-type coolers symmetrically spaced around the stator frame.

Power for the operation of the pump motors is supplied from the Tracy switchyard adjacent to the pumping plant, through circuit breakers connected to a 13·8 kV bus. Located in the switchyard and connected to the same bus is a 50,000 kVA synchronous condenser.

Construction of the Tracy pumping plant began in August 1947, and was completed in December 1949. Under a separate contract installation of the motors and pumps and electrical and mechanical equipment was started in September 1949 and completed in April 1951. Construction of Tracy switchyard was started in December 1949, and was completed in November 1951.

(To be continued)

Resistance Wires and Tapes. A booklet received from the Vactite Wire Co. Ltd., lists the various products they manufacture, giving dimensions, physical characteristics and spheres of application with a view to assisting the designer of apparatus in his work.

The Stability of Throttled Surge Chambers

The following notes by Prof. L. Escande, of l'Ecole Nationale Superieure d'Electrotechnique et d'Hydraulique, Toulouse, were recently presented to l'Academie des Sciences de Paris by M. Charles Camichel

Note 1: Oscillations in a throttled surge chamber not conforming to the conditions of stability enunciated by Thoma. Theory established during a study of the surge chamber of the Yablanica station, Yugoslavia.

When a power station is feeding an electrical network under constant load, the action of the governors tends to create an oscillation of the water level in the surge chamber. On the other hand, the dissipation of energy due to the loss of head in the supply tunnel is a factor tending to damp out these oscillations. Thoma has shown that these oscillations are damped out, maintained, or amplified indefinitely according as the product $2p_oh_o$ is greater than, equal to, or less than unity.

We are about to show that in the case of throttled surge chambers this conclusion is not entirely correct; for $2p_oh_o<1$ the oscillations are not amplified indefinitely but are merely maintained at a stable value.

First, let us consider a shaft in which, instead of an actual constriction giving rise to a head loss according to the square law $r=r_ov^2$, there is a hypothetical constriction in which the head loss is in the form $r=mv$ where m is a constant. We will examine the case of operation at full load, the turbine absorbing the maximum stable discharge Q_o. Let us set out the following fundamental equations, using standard notation:—

$$v\frac{dv}{dz} + \frac{1}{2\pi}\frac{du}{dt'} + z \pm p \pm r = 0$$

$$z = x - p_o,\ p = p_ow^2,\ r = r_ov^2 \quad \text{.....(1)}$$

$$u = 1 - \frac{p_o}{h_o} - \frac{z}{h_o} \mp \frac{r}{h_o}$$

and let us substitute $r=mv$ for $r=r_ov^2$.

From this we deduce, using the same approximations as in Thoma's theory, the following linear differential equation of the second order, with constant coefficients:—

$$(h_o - m)\frac{d^2x}{dt'^2} + 2\pi[2p_oh_o - 1 + m(h_o - 2p_o)]\frac{dx}{dt'} + 4\pi^2(h_o - 2p_o)x = 0$$

Let us confine ourselves to the case $2p_oh_o<1$ for surge chambers not satisfying Thoma's conditions, and let us take account of the conditions

$$h_o>1, \quad h_o - m>0, \quad h_o - 2p_o>0$$

which are always found in practice. The solution of the differential equation will then give the following results:—

for $m<r_1$: oscillations indefinitely amplified

for $m=r_1$: oscillations maintained at a cycle period of $\sqrt{\frac{h_o^2 - 1}{h_o - 2p_o}}$

for $r_1<m<r_2=r_1 + \frac{2(h_o - 1)}{h_o - 2p_o}$: damped oscillations

for $m>r_2$: dead-beat condition in which

$$r_1 = \frac{1 - 2p_oh_o}{h_o - 2p_o} \quad \text{.....(2)}$$

From this general discussion we would merely make use of the following relationships: that the amplitude of the oscillations

is indefinitely amplified when $r<r_1$
is damped and dies away when $r>r_1$
remains constant when $r=r_1$

Now, the hypothetical constriction imposes a limit corresponding to a loss of head $r=r_1v$, and gives rise to a damping effect which, associated with that causing the loss of head in the supply tunnel, exactly balances the amplifying action of the governor; this results in a pure and simple maintenance of the oscillations without amplification or damping.

Let us return to the actual constriction $(r - r_ov^2)$ giving rise to a loss of pressure $rv=r_ov^3$ and compare its action with that produced by the hypothetical constriction limit $(rv=r_1v^2)$ corresponding to the maintenance of the oscillations. If a small oscillation occurs, curve C (v, r_ov^3) in Fig. 1, being below curve

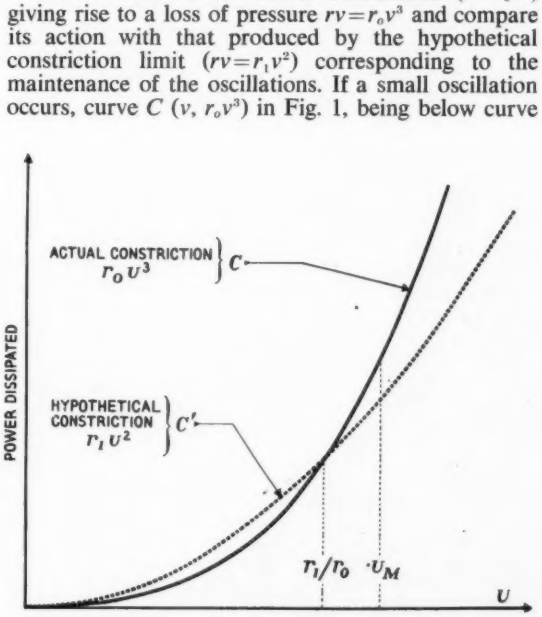


Fig. 1

C' (v, r_1v^2) beneath the point of intersection (r_1/r_o, r_1^3/r_o^2), the loss of energy in the actual constriction is insufficient to neutralise the action of the governor. The amplitude of v progressively increases, passes r_1/r_o the abscissa of the point of intersection of the two curves C and C', and stabilises at a value v_m such that during each cycle the loss of energy in the actual constriction is the same as that in the hypothetical constriction already considered:—

$$\int_o^T r_o v^2 dt = \int_o^T r_1 v^2 dt \quad \text{.....(3)}$$

On the other hand, commencing with an initial oscillation of high amplitude and considering the energy balance in the same manner, it can be established that damping occurs until the condition of stable oscillation just defined is reached.

If, on a first approximation, the oscillations are assumed to be sinusoidal ($v = v_m \sin \omega t$), the energy balance expressed in equation (3) gives:—

$$v_m = 1{\cdot}18 \frac{r_1}{r_o} = 1{\cdot}18 \frac{1 - 2p_o h_o}{h_o - 2p_o} \quad \text{.....(4)}$$

Thus, in a throttled surge chamber having a section less than the limiting value given by Thoma, the foregoing theory points to the existence of stable oscillations having an amplitude v, given, on a first approximation, by equation (4).

Note 2: Graphical determination of the amplitude of stable oscillations of the water level in a throttled surge chamber not complying with Thoma's conditions; approximate expression for this amplitude.

In the foregoing note we have demonstrated anal-

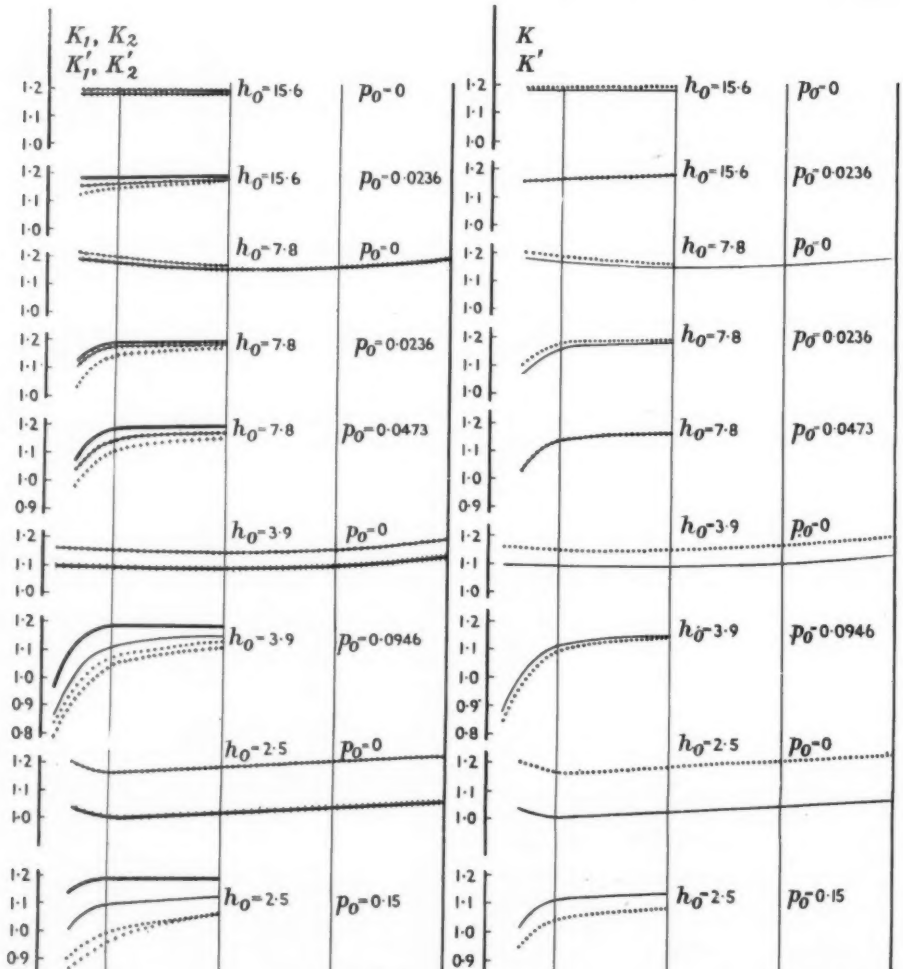


Fig. 2

tically the existence of stable oscillations of the water level in throttled surge chambers having a section below the limiting value laid down by Thoma.

Seeing that certain assumptions were made in the calculations—in particular that the loss of head p in the supply tunnel was linear—it is important to prove that the results obtained remain valid not only for small oscillations but equally for large ones.

We have been able to obtain this result by means of a graphical method based on equations (1) in our previous note, and the systematic study of 44 examples has enabled us in every case to verify the existence of a stable condition of maintained oscillations.

These oscillations are not exactly sinusoidal, and this leads us to lay down the following equations:—

$$x_{1m} = -K_1\frac{r_1}{r_o},\quad x_{2m} = K_2\frac{r_1}{r_o},\quad K = \frac{K_1+K_2}{2}$$

$$v_{1m} = -K_1'\frac{r_1}{r_o},\quad v_{2m} = K_2'\frac{r_1}{r_o},\quad K = \frac{K_1'+K_2'}{2}$$

where v_{1m}, v_{2m}, x_{1m}, x_{2m} are the maximum amplitudes of the negative or positive alternations of v and x, the values being related to the size of the water surface above the stable dynamic level corresponding to the maximum discharge Q_o.

Fig. 2a gives the values of K_1, K_2, K_1', K_2' and Fig. 2b those of K and K'.

Within the limits obtaining in industrial applications, the values obtained are all between 1 and 1·8, and generally are much nearer to the second figure than to the first. We can thus retain the approximate expression:—

$$x_m = x_m \,.\, Z_X = 1{\cdot}18\frac{r_1}{r_o}Z_X = \frac{1{\cdot}18}{r_o}\,.\,\frac{1-2p_oh_o}{h_o-2p_o}\,.\,Z_X$$

for the amplitude of the oscillations, as the error, which is always in excess of the true value, is invariably below 15 per cent. and is always on the side of safety.

We have also studied the case of a steady condition corresponding to a fraction $\frac{Q_o}{n}$ (n being greater than 1) of the maximum discharge Q_o. Stable oscillations are obtained the amplitude of which is given by the approximate expression:—

$$X_m'' = \frac{1{\cdot}18}{r_o}\,.\,\frac{Z_X}{n}\,.\,\frac{1-2p_oh_o\left[1+\frac{p_o}{h_o}\left(1-\frac{1}{n^2}\right)\right]}{h_o+p_o\left(1-\frac{3}{n^2}\right)} < \frac{X_m}{n}$$

This amplitude is smaller, both absolutely and proportionately, than that apertaining to full flow.

Industrial Consequences of the Preceding Results

For throttled surge chambers it becomes possible to adopt, with complete safety, a section smaller than the limiting value laid down by Thoma on condition that a procedure is adopted of temporarily subordinating the electrical output to the hydraulic pressure eliminating the oscillations. A failure to carry out this procedure would not be accompanied by any major inconvenience, as the oscillations set up would have a limited amplitude instead of growing indefinitely as in an ordinary surge chamber.

Trief Slag-Cement Process for Glen Moriston

The North of Scotland Hydro-Electric Board, in an attempt to reduce constructional costs and save cement, have decided to use ground blast-furnace slag in the construction of the Loyne and Cluanie dams of the Glen Moriston hydro-electric scheme in Inverness-shire.

The process which the Board have decided to try out is called the Trief process and the contract for the construction of the dams has been placed with Mitchell Engineering Co. Ltd., of Peterborough, who have themselves also carried out investigations.

In 1948 the attention of the Hydro-Electric Board was drawn to the use being made of ground blast-furnace slag in the construction of a large dam by the French Electricity Board at Bort-les-Orgues in Central France. The blast-furnace slag was used in part replacement of Portland cement and the reports received were so encouraging that it was decided to send to France a deputation consisting of two engineers and a chemist to get fuller details. The process was attractive to the Board for two reasons: it held out the hope of economies in constructional costs and it might also save a considerable tonnage of cement which would become available for the export market.

The Trief process, which was patented by M. Victor Trief, a Belgian, consists of using blast-furnace slag (which is at present virtually a waste product) by grinding it wet at site in a rotary grinder. The slurry so produced is passed direct into the concrete mixers to be mixed with the appropriate quantities of Portland cement, aggregate and sand, to produce a concrete equal in strength to a similar mixture using wholly ordinary Portland cement. The French also claim that the slag cement concrete produces less heat when setting and also is more resistant to acid peaty waters than pure Portland cement concrete.

The blast-furnace slag available from Scottish steelworks contains not quite so much lime as the slag the French used, but the Highland experiment should nevertheless produce results which will be of value not only to the Board and to the country's export trade, but for other large dams at home.

The Cluanie dam is to be 112 ft. high and 2,165 ft. long and the Loyne dam 58 ft. high and 1,745 ft. long. The proportion of slag cement to Portland cement will probably be about one-third Portland and two-thirds blast-furnace slag, in which case the saving in Portland cement will amount to about 20,000 tons. In the construction of the dams, the Mitchell Engineering Company are also adopting another idea which was tried out at Bort, which is not to use shuttering but to use instead precast concrete slabs for the faces of the dams. By this means, neither steel nor timber shuttering will be needed and the demand therefore for these two scarce materials will be reduced.

Thew Lorain. A brochure in colour received from The Thew Shovel Company illustrates some striking examples of the firm's shovels and cranes in action in diverse conditions. The brochure also gives the firm's annual report and calls attention to the very satisfactory increase in the net annual sales during 1951.

Fig. 10. Benevean dam and intake viewed from upstream

The Glen Affric Scheme

In this article we describe the construction of the Mullardoch-Benevean tunnel, the Benevean dam, and the Benevean-Fasnakyle tunnel.

PART TWO

THE Mullardoch-Benevean tunnel,* which, as stated in our previous article, is 5,738 yards long and of 15 ft. 9 in. equivalent diameter, falls 3 ft. from Mullardoch to Benevean, and is unlined except for a concrete invert throughout and full-section lining at the portals and at certain sections of weak rock. Owing to the local topography it was not feasible to construct adits, and the tunnel was driven from the two ends.

The equipment used at Mullardoch may be taken as generally representative of that employed at the Benevean face and also at the various faces in the Benevean-Fasnakyle tunnel. Compressed air was derived from three 850 cu. ft. per min. compressors delivering at 100 lb. per sq. in., surplus air being used as available on the dam. A 6 in. steel main with Victaulic joints led into the tunnel, and was provided with valves at suitable points, the face valve being carried forward continuously to enable air to be shut off rapidly in an emergency. Two 2 in. air hoses with Unicone couplings led to the drilling-carriage manifolds, from which 1 in. hoses led to the drifters. A 2 in. air-driven Hayward Tyler pump supplied water from the loch at high pressure to the drills. Ventilation was provided by a 3,500 cu. ft. per min. fan blowing through 18 in. ducting, reversible gates being fitted to enable blasting fumes to be extracted. A lighting supply was brought in at 440 V and was stepped down to 110 V at intervals of 1,500 ft. by 3 kVA transformers, 60 W lamps being disposed at intervals of 30 ft.

The rock was a tough and abrasive micaceous schist, ranging from pelitic to psammitic. In the earlier part of the contract, drilling was accomplished with Ingersoll Rand 3½ in. jumbo-mounted drifters. The jumbo carriage ran on an 8 ft. gauge track, and incorporated a gauge ring to enable underbreak and overbreak tc be observed immediately and the alignment of the ring holes adjusted accordingly. Hollow round drill steel of 1¼ in. diameter was used in conjunction with two types of cruciform detachable bit. The first type was screwed to fit into the end of the steel and could be sharpened three or four times, losing ⅛ in. diameter each time. The later type had an internal corrugation which fitted over an oval spigot on the rod and locked itself on rotation. This bit could not be resharpened, but it was cheaper, the drill steel lasted longer, and it was easier to re-form the end of the steel when worn.

In the Mullardoch-Benevean drive trouble was experienced with the normal bits in certain sections of the rock, and a change was made to Atlas-Diesel equipment, consisting of light high-rotary-speed jack-

* A detailed account of the driving of the two tunnels on this scheme, but referring more particularly to the Benevean-Fasnakyle tunnel, was given in a paper, "The Mullardoch-Fasnakyle-Affric Tunnels" presented by G. G. Dillon, B.E., A.M.I.C.E., to the Institution of Civil Engineers on February 28, 1950.

hammers mounted on pneumatically extending legs and using tungsten-carbide-tipped Swedish drill steels. During the seven months in which this equipment was used, the average rate of advance was 7½ per cent. greater than during a similar period immediately prior to its adoption, but this is not regarded as a reliable comparison because the type of rock encountered varied considerably. It would have been possible to have saved six men on the drilling crew, but as the same men did the mucking the number was reduced by only three to avoid disorganising this operation. A saving in air consumption was secured of roughly 50 per cent. In the case of a face equipped from the outset on this system a light drilling carriage can be used which is cheaper and can be handled more easily than the normal heavy jumbo, but in this instance the jumbo originally used for the drifters was retained.

The drilling round was 9 ft. in depth and consisted of 57 holes arranged with a five-hole burn cut and 1 sec. delays up to a maximum of 9 seconds, giving an average pull of 8 ft. Polar ammon-gelignite A was used as the explosive for most of this drive, the amount varying considerably according to the rock encountered, but in the latter stages a large proportion of dynamite had to be used.

Mucking was accomplished by two Eimco shovels loading into 2 cu. yard skips which were withdrawn in rakes of six by Simplex diesel locomotives.

In several sections of the tunnel the rock was faulty and a lining had to be inserted. Most of these sections were lined with concrete, but two sections had to be supported by steel arches in addition to the concrete lining.

A view of the intake portal taken just before ponding commenced appears in Fig. 11. At a distance of 330 ft. from the intake portal a 120 ft. gate shaft was sunk and equipped with a 10 ft. × 11 ft. free roller control gate and a 10 ft. × 11 ft. 10 in. free rolling emergency gate. Both these gates are electrically operated and were supplied by Glenfield & Kennedy Limited. Immediately upstream of these gates is a venturi section designed to function in conjunction with a British Pitometer instrument to indicate and record the flow through the tunnel.

Fig. 12. Discharge tunnel and turbine draught tube of Mullardoch subsidiary generating station

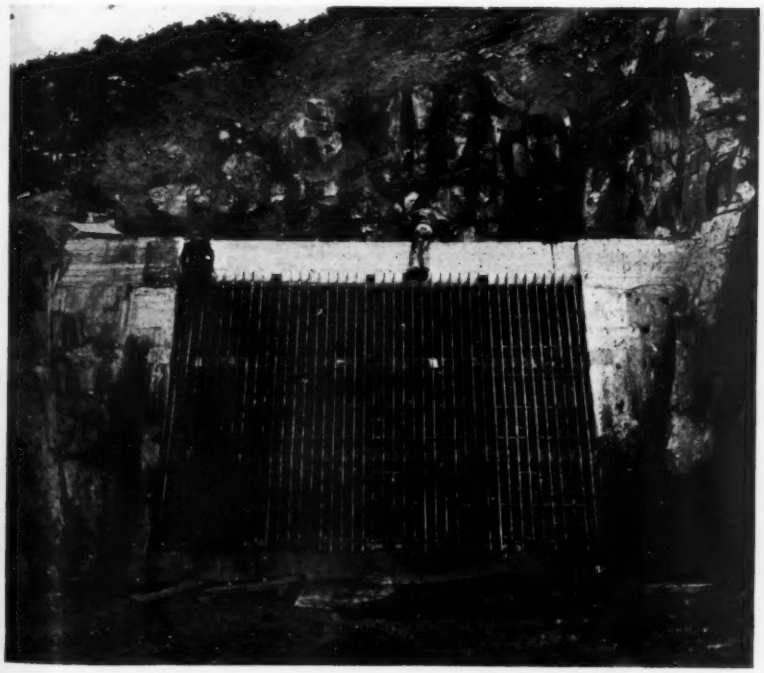


Fig. 11. Mullardoch tunnel intake portal before ponding

In order to make use of the difference in level between the two lochs—about 80 ft.—an underground subsidiary generating station is being constructed near the tunnel intake. The intake for this station is 200 ft. downstream of that of the main tunnel, and is of rectangular section with a semicircular roof, 10 ft. wide by 9 ft. 5 in. high. A venturi section is incorporated. On the discharge side the tunnel has been made of the same section as the main tunnel, enabling the same driving equipment to be used, and both tunnels have been concrete lined up to a point beyond the junction. A 20 ft. by 12 ft. access and gate shaft has been sunk to the power-house chamber, which is 32 ft. long, 26 ft. wide and 25 ft. high.

A vertical Francis turbine by Gilbert Gilkes & Gordon Limited will be installed here, flexibly coupled through gears

Fig. 13. Benevean dam in the early stages of construction, showing the river diversion and excavation of the original channel in progress. The batching plant is on the far hillside to the right

to a horizontal Bruce Peebles induction generator of 2,400 kW capacity—believed to be the largest machine of this type to be installed in this country. All mechanical and electrical controls for this set will be brought up to the winch house at the top of the access shaft.

Seeing that the main tunnel will be flooded for some considerable time before this station is ready, measures have been taken to isolate the station until it is complete. Over the mouth of the station intake a rectangular reinforced-concrete bulkhead has been fitted. It is temporarily latched in position and will be secured by the outside water pressure when the loch is filled, but it is inclined forward so that it will drop out when the intake tunnel is flooded. On the discharge side the draught tube, which is of steel, is fitted with a blank flange which can be dropped when desired.

The main tunnel outfall at Loch Benevean is arranged so that the tunnel can be drained for inspection, and to this end it is equipped with a Glenfield & Kennedy 10 ft. by 14 ft. 1 in. fixed roller gate with hand operating gear and incorporates a pump shaft communicating with the tunnel just behind the gate. A screen protects the offtake to the pump shaft, and four Beresford 120,000 g.p.h. portable submersible pumps are held at Fasnakyle power station.

Benevean Dam

Benevean Dam is 582 ft. long and 125 ft. high from cut-off trench to spillweir; is of the mass-concrete gravity type and involved the excavation to payment lines of 20,000 cu. yards of rock and the placing of 54,000 cu. yards of concrete. Drawings of the dam are given in Fig. 14, and various views in Figs. 10, 13 and 15. The spillway extends the entire width of the dam, except for the gatehouse, and as in the case of the Mullardoch dam, the final design of the spillway channel and stilling pool was determined by model tests. A series of steps controls the flow of water to the stilling pool, which is provided with two longitudinal weirs and a transverse weir, as seen in Figs. 14 and 15. A 6 ft. scour culvert is incorporated in the dam opposite the stilling pool, and is equipped with a 72 in. to 60 in. needle valve with jet disperser and an electrically operated free rolling emergency gate, 6 ft. span by 7 ft. deep. Compensation water is bled off from the scour culvert by an 18 in. electrically operated control valve. All this equipment is of Glenfield & Kennedy manufacture.

Some care was needed in preparing the dam foundations because a zone of shattered ground was found on the south side, and the rock in the river bed was open jointed and liable to shattering when blasted. The cut-off trench ranged from 5 ft. to 10 ft. deep,

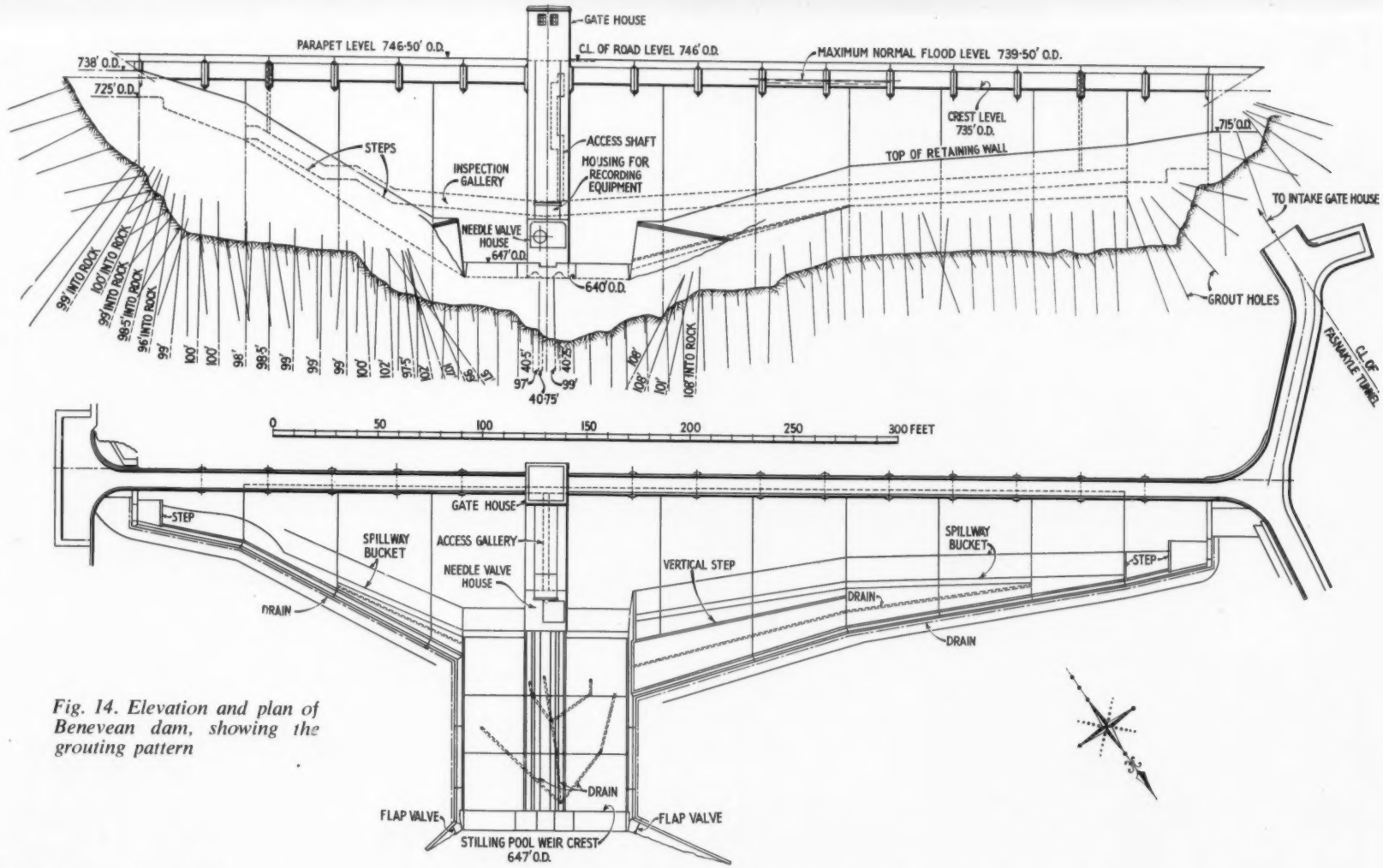


Fig. 14. Elevation and plan of Benevean dam, showing the grouting pattern

and primary grouting holes, 10 ft. deep and 10 ft. apart, were grouted under a nominal pressure of 10 lb. per sq. in. to fill the open fissures. Further holes 40 ft. deep were drilled midway between the first series and piped up through the concrete as the foundations were placed. When about 10 ft. of concrete had been laid these holes were grouted at pressures ranging up to 75 lb. per sq. in. In the river bed and throughout the shattered zone on the south bank, inclined holes 100 ft. deep were set down from the upstream step of the cut-off trench and grouted at 120 lb. per sq. in. —a value greater than the final hydrostatic pressure. Altogether, upwards of 50 tons of cement was expended in these operations. The grouting pattern is shown in the dam elevation in Fig. 14.

Aggregate for the dam was derived from suitable spoil from the Benevean-Fasnakyle tunnel. As will be described later, two adits were driven in opening out the tunnel, and a spoil dump was established midway between the two adits. A crushing and grading plant reduced this material to $2\frac{1}{2}$—$1\frac{1}{2}$ in. and $1\frac{1}{2}$—$\frac{3}{16}$ in., these sizes being taken by lorry to a stockpile at the dam. A second crushing plant—also to be described later—was erected at Fasnakyle to treat a deposit of river gravel, making $2\frac{1}{2}$—$1\frac{1}{2}$ in., $1\frac{1}{2}$—$\frac{3}{4}$ in. and $\frac{3}{4}$—$\frac{3}{16}$ in., and a crushed sand (minus $\frac{3}{16}$ in.). Crushed gravel was used for the 4 : 1 facing concrete and tunnel spoil for the 7 : 1 hearting.

A batching plant was set up on the north bank of the river a little way downstream of the dam, and was equipped with two 1 cu. yard Stothert & Pitt mixers. A Bailey bridge was built across the site immediately downstream of the dam, and the skips were handled by a travelling derrick on the north side and a steam derrick on the south side. Bottom-opening skips of 2 cu. yard capacity were used, and were brought from the batching plant by diesel locomotives.

The dam was divided into 14 blocks, nine being 45 ft. long and the others ranging from 20 to 51 ft. and the system of shuttering and general procedure were similar to those employed at Mullardoch. Low-heat cement was used for the north side of the dam, but owing to rising costs, ordinary Portland cement was used for the south side. The speed of placing was carefully regulated to prevent the mass temperature rising above 100°F. Great care was taken with scabbling. At an optimum period after placing—which ranged from 4 to 15 hours with the Portland cement

Fig. 15. View of the Benevean dam, looking upstream, showing the temporary openings and the completed stilling pool in the foreground

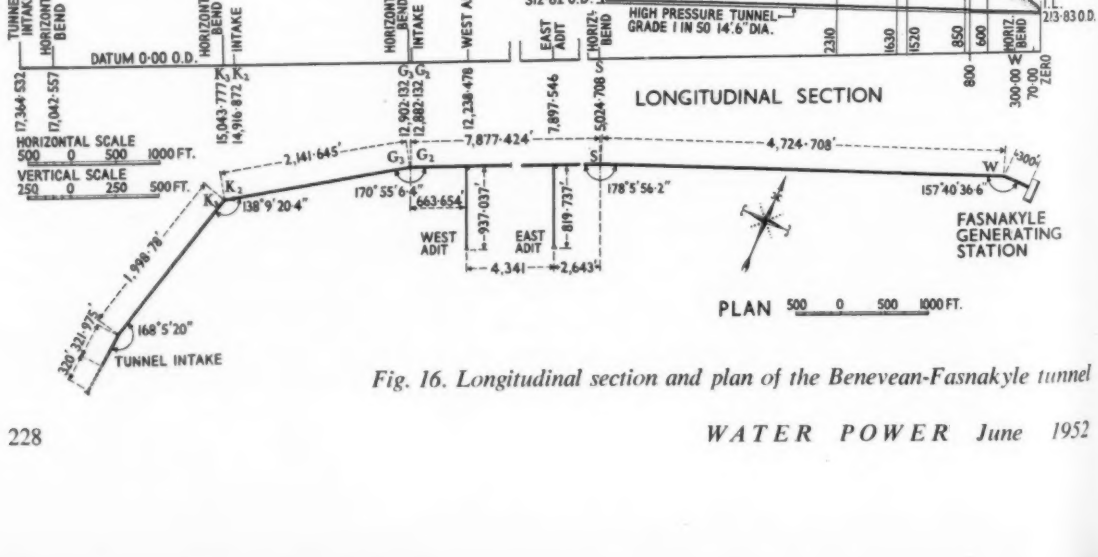


Fig. 16. Longitudinal section and plan of the Benevean-Fasnakyle tunnel

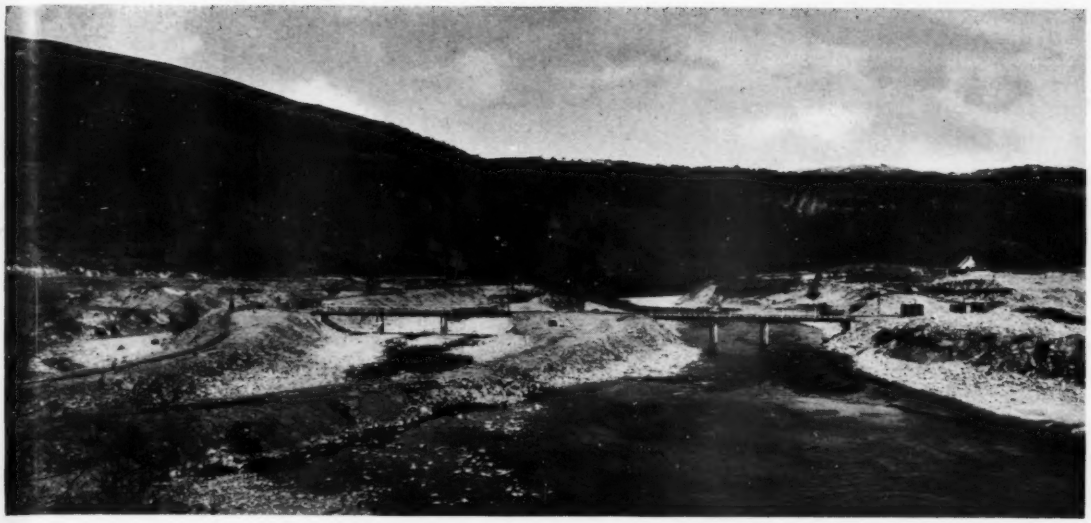


Fig. 17. Downstream view of the river diversion at Fasnakyle. The site of the generating station is on the extreme left, and that of the crushing and dry-batching plant off the picture to the right

and from 6 to 24 hours with the low-heat cement—the surface was thoroughly scoured with high-pressure hoses, and brushed until the exposed aggregate was completely cleared of cement. The use of picks was discouraged in order to avoid a layer of shattered concrete. Before placing the next lift a ½ in. layer of 2:1 cement mortar was laid over the surface. This care was justified by the event, for when the loch was filled the dam showed no sign of seepage.

The northern half of the dam, comprising blocks 9 to 14, was carried up first, as it was clear of the river and stood on satisfactory ground. To enable blocks 5, 6 and 7 spanning the river to be commenced a diversion channel was formed through the site of block 8. Three temporary openings were left, two in block 5 (see Fig. 15) and one in block 7, and the scour culvert was set in block 6, the river then being diverted back through these openings to enable block 8 to be proceeded with. Finally, during a dry, frosty period, one of the openings was closed in the dry, the others being sealed by temporary gates while they were being filled in.

The Benevean-Fasnakyle Tunnel

A plan and section of the Benevean-Fasnakyle tunnel are given in Fig. 16. A low-pressure section, of 14 ft. 6 in. equivalent diameter, extends a distance of 12,340 ft. to a vertical shaft, 341 ft. deep, leading to a circular high-pressure tunnel, 14 ft. 6 in. in diameter, 4,224 ft. long, and falling at a gradient of 1 in 50. A surge shaft 150 ft. high and 45 ft. in diameter surmounts the vertical shaft. Finally, the high-pressure tunnel divides into three steel-lined tunnels, each 800 ft. long and 8 ft. 4 in. in diameter leading to the three generating sets.

Two side-stream intakes communicate with the low-pressure section of the tunnel, and a third stream discharges into the surge shaft.

Work on the low-pressure section was greatly expedited by the construction of two adits at 5,126 ft. and 9,582 ft. from the intake respectively, and by opening up two faces from the west adit and one face, travelling east, from the east adit. Thus, including the drive from the intake, there were four faces at work in the low-pressure tunnel and one in the high-pressure tunnel (disregarding the three pipe tunnels) which was driven from east to west, the longest single drives being the high-pressure tunnel and the low-pressure section between the west and east adits.

Shafts were driven 120 ft. upwards to form the side-stream intakes, and on the site of the surge shaft a 9 ft. square pilot shaft was sunk and the surge shaft glory-holed out to full diameter when the drive from the east adit had been completed. The shaft between the low-pressure and high-pressure tunnels was cut by a combination of sinking and driving upwards.

The equipment for driving the tunnels was generally similar to that employed for the Mullardoch-Benevean tunnel except that drifters were employed throughout.

Some care was needed in cutting the entries of the three high-pressure pipe tunnels, because they passed imediately below the Cannich Glen-Affric road, which had to be kept open. Working space, also, was restricted by the proximity of the generating-station site, and it was necessary to run the 2 ft. gauge service tracks along the intervening space between the road and the station site, curving the tracks at a small radius to enter the tunnels.

In order to lay the foundations of the generating station the river had to be diverted, and cofferdams were built to direct the river into a temporary channel cut to the right of the normal bed, as seen in Fig. 17.

Unlike the Mullardoch-Benevean tunnel, the whole of the Benevean-Fasnakyle tunnel has been lined. In the low-pressure section the lining is 2 in. over points and 8 in. to payment line, and in the circular high-pressure section 10 in. over points and 16 in. to payment line. Originally a thin steel lining was considered for the high-pressure tunnel, but it was feared that in the event of the tunnel being dewatered, any hydrostatic pressure that might have built up around the tunnel might collapse the lining. Seeing that the rock

was a sound mica schist it was decided to rely on it for impermeability and strength assisted by a comparatively thin concrete lining and by grouting. For the last 700 ft. from the power station the rock cover decreased rapidly, and it is at 100 ft. upstream of this point that the tunnel has been trifurcated, each limb being provided with a steel lining capable of withstanding all possible internal and external pressures.

Material for the concrete lining of the high-pressure tunnel was obtained from a gravel deposit on the right-hand bank of the river, and a crushing and grading plant was set up on the bank opposite to the power-station site. A Ruston-Bucyrus 37B shovel was used to dig the gravel, which was crushed to produce a graded aggregate $2\frac{1}{2}$ — $1\frac{1}{2}$ in., $1\frac{1}{2}$ — $\frac{3}{4}$ in. and $\frac{3}{4}$ — $\frac{3}{16}$ in. and a washed sand (minus $\frac{3}{16}$ in.). A batching plant was set up adjacent to the crushers, the dry batches being taken to the tunnel over the temporary bridges seen in Fig. 17 and mixed in two 1 cu. yard mixers located just outside the tunnel portals. Sand and crushed gravel for part of the Benevean dam were also obtained from this source.

Fig. 18. Travelling shutter, Pressweld placer and concrete plant assembled outside the Fasnakyle tunnel

Fig. 19. High-pressure section of Fasnakyle tunnel being lined. Note the grouting holes in the tunnel walls

Aggregate for the lining of the low-pressure section of the tunnel was derived from the spoil heap and crushing plant which, as mentioned in our previous article, were established midway between the east and west adits, and provided graded psammitic schist for the Benevean dam and the road diversion. For the tunnel lining a batching plant was erected adjacent to this crushing plant, the dry batches being taken in skips by diesel locomotives through either the east or the west adit according to circumstances.

Fig. 18 is a view of the equipment used to line the low-pressure section of the tunnel, consisting of a travelling steel shutter, a Pressweld placer, a 1 cu. yard mixer, and a conveyor belt. The shutter, designed by Cyril

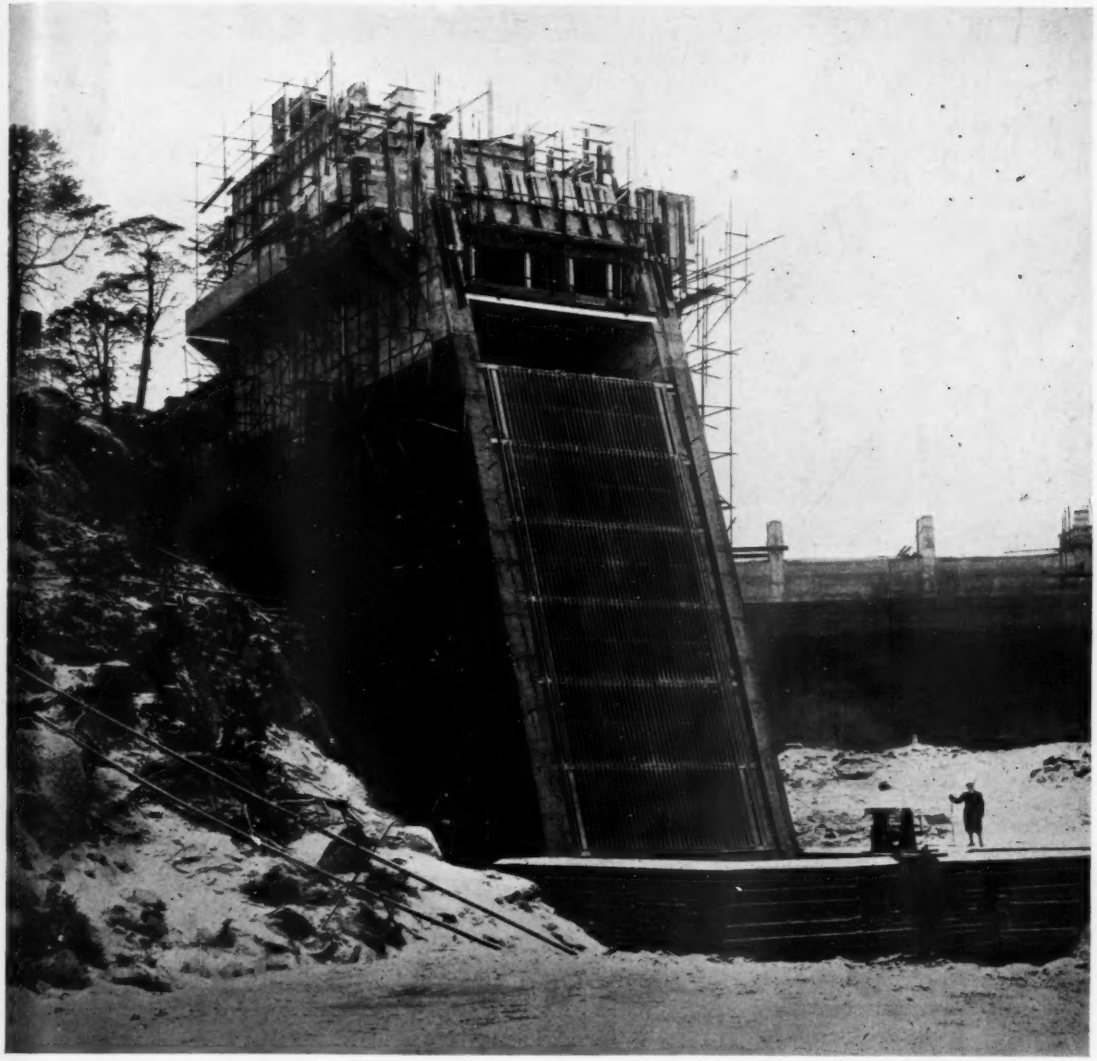


Fig. 20. Fasnakyle tunnel intake structure, showing the gatehouse under construction. Benevean dam can be seen in the background

Parry, was 85 ft. long and set the sides and arch in one operation. The invert was cleaned by hand in advance of the shutter and a rough layer of concrete placed to carry the shutter sleepers, the invert itself being placed after the shutter had passed. To save time, all these operations proceeded simultaneously in different parts of the tunnel.

One of the most interesting features of the lining operation was the introduction of the Pressweld placer. Dry mix for a cubic yard of 4 : 1 concrete, having a maximum aggregate size of 1½ in., was tipped into the boot of the conveyor and transferred thereby to the mixer, which discharged the completed mix to the placer. From the placer a 6 in. steel pipe extended over the crown of the shutter, as seen in Fig. 18, and the concrete was blown through this pipe by compressed air, lightly vibrated to induce flow off the crown of the shutter, and punned through side hatches. Mr. Dillon, in his paper referred to earlier in this

Fig. 21. Trifurcation point and bellmouths of the high-pressure pipes in Fasnakyle tunnel

article, stated that the machine was capable of placing up to 30 cu. yards of concrete per hour and required about 1,000 cu. ft. of compressed air per minute at a pressure of 100 lb. per sq. in. He also gave the cleaning and setting time for the shutter as four to five hours, the placing time as six hours, and the striking time as 18 hours. The desirability of reducing the striking time was considered to be evident.

Lining proceeded from the intake portal to the east adit, after which the equipment was turned round and the lining completed from the surge shaft to the east adit, this reversal being necessary to keep the conveyor boot addressed to the transport from the adits.

Concrete for the surge shaft and high-pressure shaft was also batched at the inter-adit plant, mixed at the foot of the surge shaft, and hoisted or lowered as required. The shuttering followed normal practice.

For lining the high-pressure tunnel similar equipment was used to that for the low-pressure tunnel, but the procedure differed in some respects. First, the sides were cleaned by hand and curb walls placed to take rails for a travelling bridge. This operation proceeded upstream from the trifurcation point, and cleaning and concreting took place simultaneously. The travelling bridge just mentioned was then erected to place the concrete for the invert; it was 150 ft. long and carried a double track. Work proceeded downstream from the high-pressure shaft, and was carried out in steps of 150 ft. corresponding with the length of the bridge. The invert was cleaned and concreted to 34° on either side of the vertical centre-line, the concrete surface being shaped by a 5 ft. drag screed carried under the bridge.

When the invert was complete, a track was laid on it to carry the travelling shutter and the Pressweld placer, but in this case a conveyor belt was not required as the travelling bridge used for the previous operation was available to bring the skips over the placer. The shutter for this tunnel was 60 ft. long and took its location from the invert.

One of the objections that have been advanced against the Pressweld placer is that the concrete is not placed horizontally but follows its angle of repose. In consequence, it is contended, there is a risk of segregation of the aggregate and of the formation of cold joints. Whether this objection is valid in practice is as yet a subject of controversy, but in order to avoid any risk of trouble on this score, the concrete in the high-pressure tunnel up to the horizontal centre-line was placed by hand, the remainder being placed by machine.

The final operation was grouting, which was carried out by the Cementation Co. Ltd. At longitudinal intervals of 5 ft., eight steel pipes, 12 in. long and 2½ in. in diameter, were placed in the concrete. The roof was cavity grouted immediately upon completion, and alternate grouting tubes were drilled through to a depth of 10 ft. and left to bleed for one month, after which they were grouted at 100 lb. per sq. in. The remaining holes were then drilled to a depth of 14 ft. and three months after the concrete had been placed were grouted at 300 lb. per sq. in.—a value greater than the hydrostatic pressure.

At the trifurcation point the tunnel was belled out to 24 ft. diameter and the mouths of the three steel-lined tunnels brought out on 120° axes, as seen in Fig. 21. A gully was formed in the floor of the bell-mouth leading to three 12 in. scour pipes.

The steel lining for the tunnels is 1¼ in. thick and was supplied in 8 ft. 9 in. lengths with treble-riveted butt joints by P. & W. Maclellan Limited, these lengths being welded together in pairs on site to form 17 ft. 6 in. strakes. An electric winch was located at the trifurcation point to draw the strakes in on bogies, concrete stools having been previously set to locate the strakes. An internal chamfer was cut on the ends of the pipe sections, and a watertight joint was made by caulking the chamfer with Philplug and covering it with a ⅜ in. by 3 in. steel strap which was fillet welded to the pipe wall on each side. The concrete stuffing was then filled in round the pipes. At the station the steel linings were carried forward from the tunnel portals to the station substructure, and were subsequently buried in concrete. Finally, the pipe interiors were shot-blasted and sprayed by Schori Metallising Process Limited with a layer of zinc and a layer of aluminium, followed by one coat of Bowranite bitumen red-lead paint and two coats of Super Black Finishing bituminous paint.

Returning to the intake end, the intake structure is built out into the loch, as will be seen from Figs. 10 and 20. A silt wall protects the toe of the structure, and a coarse screen of 20 ft. span by 87 ft. 6 in. deep covers the intake opening. Behind this screen is a fine screen 19 ft. span by 80 ft. deep, and both screens are provided with electrically operated rakes discharging to hand-propelled skips. A 12 ft. span by 15 ft. deep free rolling emergency gate followed by a free roller control gate of similar size control the entry of water into the tunnel. All this equipment was supplied by Glenfield & Kennedy Limited.

As already mentioned, one of the side-stream intakes flows directly into the surge shaft, but for each of the other two intakes a small dam has been thrown across the stream concerned, and the water taken through a screen to an offtake chamber on one side of the water-course. A syphon in this chamber transfers the water through a feed pipe to a vortex chamber immediately above the tunnel, a loop being introduced into the feed pipe near the syphon to aid priming. To remove any entrained air from the water before it enters the tunnel, the water is introduced tangentially into the chamber so that it forms a vortex; a vent pipe is taken from the roof of the chamber to the surface, and an aperture in the centre of the floor discharges the water to the tunnel.

Each of the adits is closed off from the tunnel by a concrete plug containing an access pipe which is normally blanked off. Provision is made at the two adits to accommodate the equipment required for flow measurement by the salt-velocity method when turbine efficiency tests are made.

(To be continued)

Hydro-Electric Projects in North Wales. In our issue of May 1952, page 169, we gave particulars of the various proposals placed before Parliament by the British Electricity Authority to extend existing hydro-electric works at Dolgarrog and Maentwrog and to construct a new scheme in the vicinity of Blaenau Festiniog. It should be added that these projects are being engineered for the British Electricity Authority by Messrs. Freeman, Fox & Partners in association with Messrs. James Williamson & Partners. At the moment of going to press the Bill is in the Committee stage following the second reading in the House.

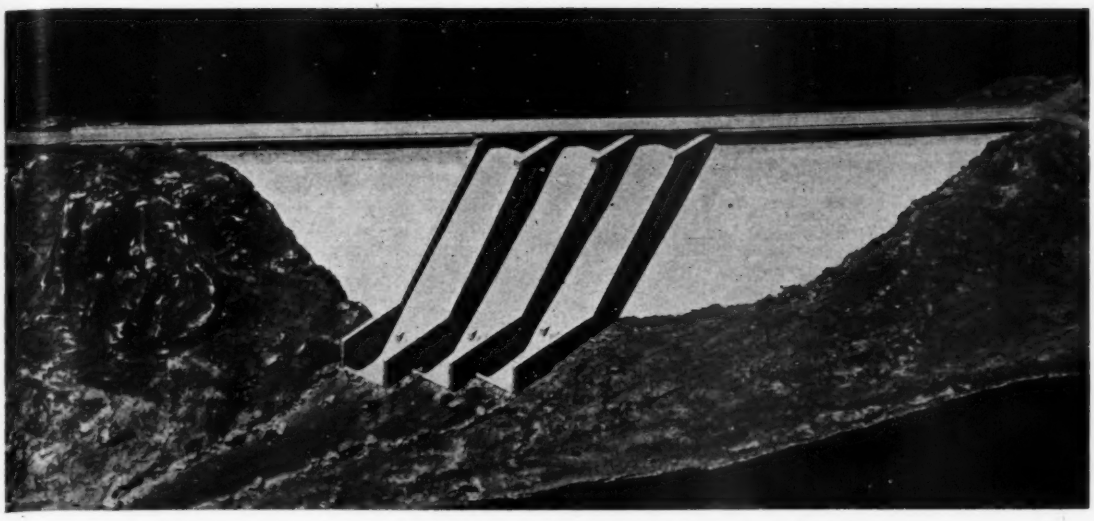


Fig. 1. Model of Los Hurones dam (spillway buckets since modified)

Developments on the Guadalquivir

Supplementary notes on the development of this river are given by S. H. Wearne to the material we published last year

A SURVEY by M. Castillo, of the Hidroeléctrica Española Company, covering Spanish dams and hydro-electric plants in service and under construction in 1950 appeared in WATER POWER in April, 1951. This indicated the large scale of the development programme in progress for 43 new dams, nearly doubling the number of 56 then in service.

Fig. 2 is based upon a recent seven-weeks' tour of fourteen sites in all parts of the river Guadalquivir. Five further new dams are shown: Cubillas, an earth dam on the river Genil near Granada; los Hurones, spillway gravity dam (Fig. 1) on the river Gaudalete; and three low-head power-house dams on the main river: Cantillana, Pedro Marin and Doña Aldonza. These last two are similar left- and right-hand designs, employing two identical 6,500 kVA sets and four 5 by 15 m. sector gates each; a project for a third similar station nearby had not been adopted. Thirteen other projects for dams of considerable size are shown on the map.

The Government have recently decreed the principle that electrical generation should be incorporated into reservoir systems originally designed for flood control or for irrigation and civic water supply. Thus small power plants will be installed at Cubillas, Guadalén and in the Bornos-los Hurones water-supply system for Cadiz. Pintado and Tranco de Beas are multi-purpose designs with more important power generation; at Pintado, capacity to the extent of 17,500 kW is being installed and a further 3,600 kW projected, and at

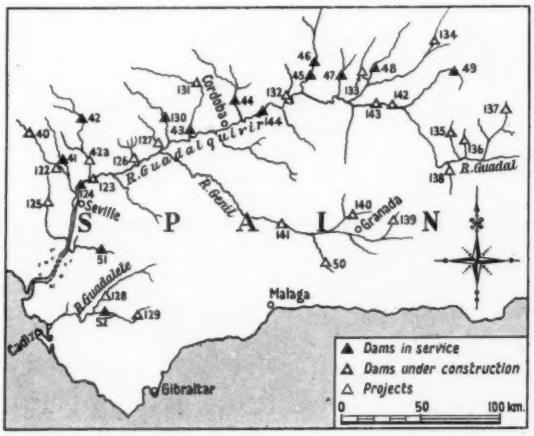


Fig. 2. Map of the Guadalquivir developments

40 Aracena
41 Cala
42-42a Pintado
43 la Breña
44 Guadalmellato
45 Encinarejo
46 Jándula
47 el Rumblar
48 Guadalén
49 Tranco de Beas
50 Bermejales
51 Torre del Aguila
52 Guadalcacin
122 la Minilla
123 Cantillana (L)
124 Alcala del Rio (L)
125 Cuervo
126 Guadalbacar
127 Retortillo
128 Bornos
129 los Hurones
130 Bembezar
131 Puente Nuevo
132 las Eguas
133 Guarizzas
134 Guadalmena
135 Guadalentin
136 Portillo
138 Negratin
139 Quentar
140 Cubillas
141 Iznajar
142 Doña Aldonza (L)
143 Pedro Marin (L)
144 el Carpio (L)

137 unnamed

L indicates low-head power-house dam.
The site numbering used by M. Castillo on his map is continued here. No. 44 appears to have been renamed.

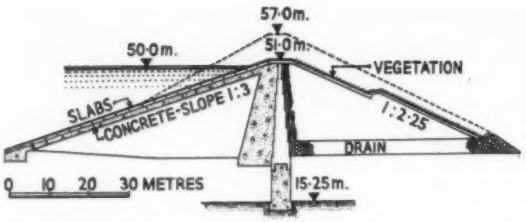


Fig. 3. Torre del Aguila dam, showing proposed heightening

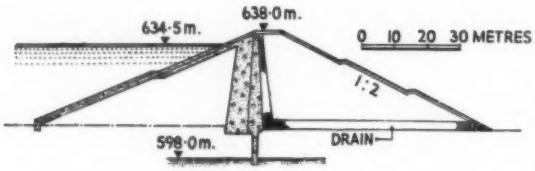


Fig. 4. Early design for Cubillas dam

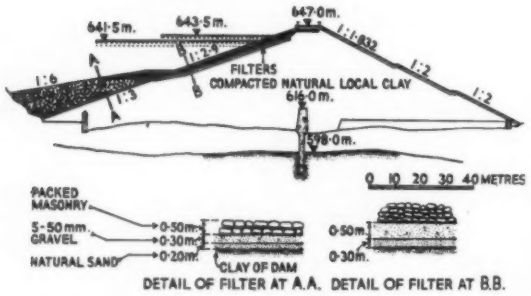


Fig. 5. Final design of Cubillas dam (third project) as now under construction

Tranco an underground power house for plant of 29,750 h.p. is under construction.

Torre del Aguila and Cubillas Earth Dams

The Torre del Aguila earth dam was one of the earliest works for flood control and irrigation storage in the valuable agricultural area of the Guadalquivir basin.

Exploratory work is now in progress on increasing the crest and water levels by six metres, as indicated in the sketch (Fig. 3) of the dam profile. According to extrapolation of the storage curve, this will increase the reservoir capacity from 80 to approximately 135 million cubic metres.

The existing lateral 'staircase'-type spillway, which consists of a regular series of rectangular stepped sills, will be unsuitable for modification for flood discharge at the increased water level, and will be abandoned. Model tests are therefore in progress on a modern type of spillway canal, having a sill formed by a weir, curved in plan, similar to that under construction at Cubillas.

The general design of Torre del Aguila has been found unsatisfactory, the heavy core being too rigid to suit the movement of the foundations under load. Following this experience, the first design (Fig. 4) for Cubillas was twice modified, and also considerably enlarged.

The dam is now well under construction to the design shown in Fig. 5. There is a small concrete cut-

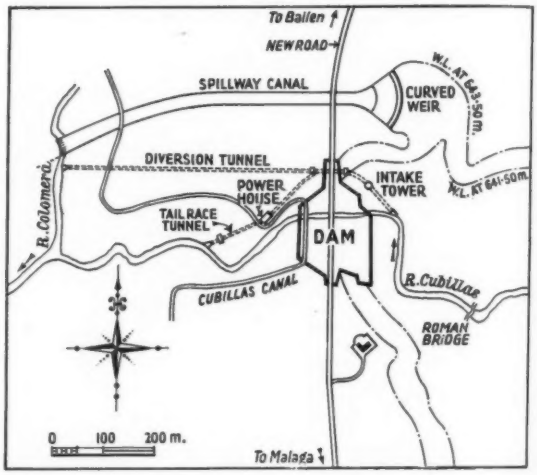


Fig. 7. Plan of Cubillas dam and spillway canal

Fig. 6. Filter construction at Cubillas

off, and the entire fill is local clay, compacted by sheepsfoot rollers. The method of protection against core erosion by reversed filters is indicated in separate detail in Fig 5, the actual construction being shown in Fig. 6.

The reservoir capacity will be 25 million cubic metres at the spilling level, which is at an elevation of 641·50 m. The maximum water level is two metres higher, the spillway being designed for a discharge of 2,000 cu. m. per sec. under this head. The greatest recorded flood is 700 cu. m. per sec.

The Cubillas reservoir is situated at the centre of a system of seven minor dams

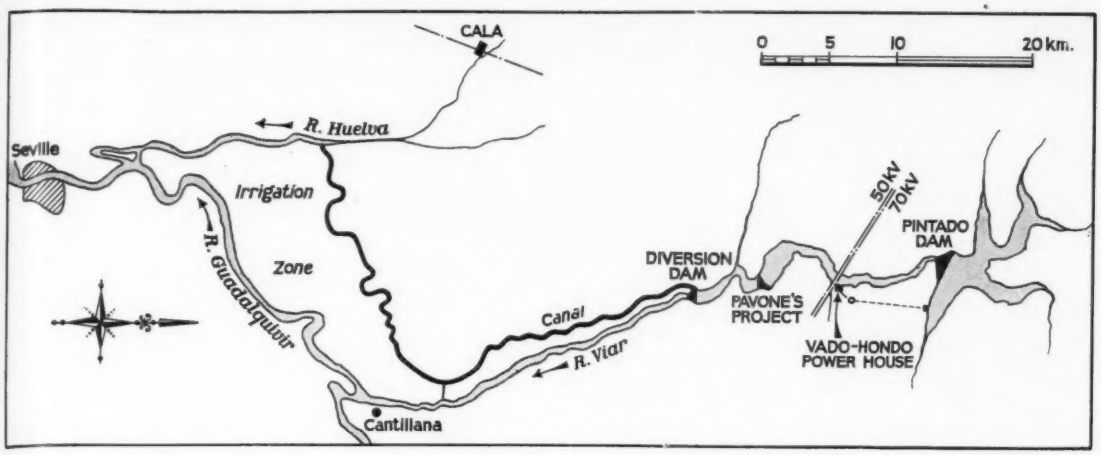


Fig. 8. The river Viar scheme

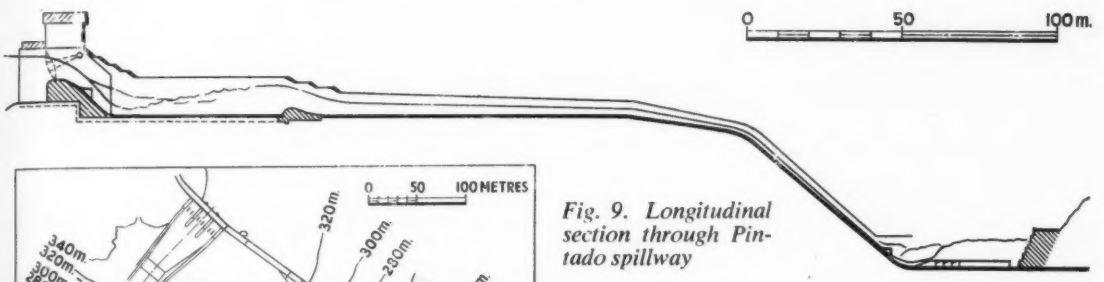


Fig. 9. Longitudinal section through Pintado spillway

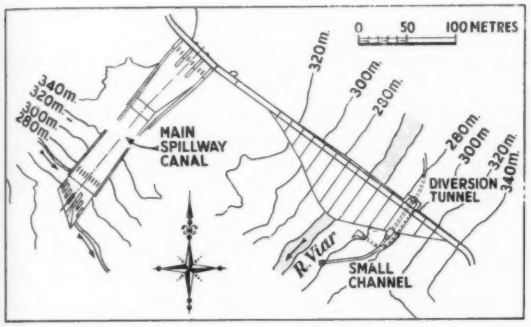


Fig. 10. Pintado dam and spillway

in an irrigation scheme for over 8,000 hectares* in the Granada-Santafe area, the reservoirs and irrigation zones being linked by canals.

A small hydro-electric plant has been incorporated into the Cubillas design by arranging that the irrigation water is discharged through a turbine. This may be bypassed if required, or alternatively the turbine discharge may be diverted to a separate tailrace tunnel when the irrigation demand is small.

The Pintado Scheme

The river Viar development, Fig. 8, is a typical multi-purpose scheme. An area of 12,000 hectares is to be irrigated, and generating plant of 17,500 kW is being installed and a further 3,600 kW projected in a system comprising one major and two minor dams.

The completed Pintado gravity dam is 81 m. high and the overall crest length 406 m. The crest was heightened 3m., without amendment to the main design, to increase the storage from 175 to 202 million cubic metres. The spillway is still under construction, and will carry discharges up to 1,400 cu. m. per sec., controlled by three Taintor gates each 12·5 m. wide, 7·5 m. high (Figs. 9 and 10).

M. Castillo's note on the extreme irregularity of flow in Spanish rivers (p.137) is well illustrated by the figures for the Pintado catchment. This has an area of 1,100 km. and an average annual rainfall of 675 mm. from which the average run-off is 175 million cubic metres per year. The maximum recorded is 514 million, and the minimum two million.

A small spillway channel on the left bank downstream of the dam will release irrigation water when the turbines at Vado-Hondo are not discharging the

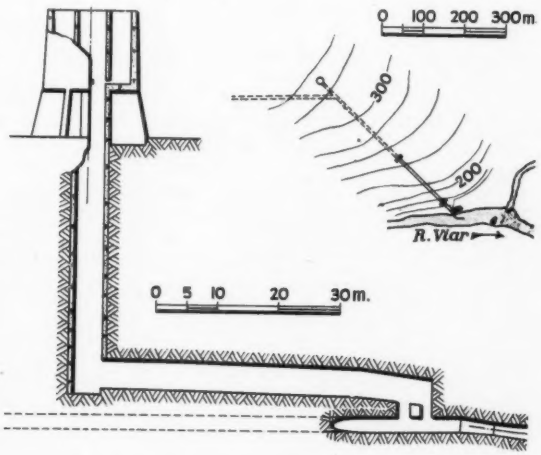


Fig. 11. The Vado-Hondo surge tank

* 1 hectare = 10,000 sq. m. = 2.49 acres.

full amount required. The flow into this channel is regulated in a small gatehouse on the downstream face of the dam. The main gatehouse on the upstream face contains a stairway and a lift to the galleries, the intake and relief gates for the spillway-channel conduit, and the control gates for the diversion tunnel.

The intake to Vado-Hondo is 2½ km. east of the main dam. The 2·5 m. diameter pressure tunnel is approximately 6·3 km. long and will pass 12 cu. m. per sec. to the turbines under a total head of 199·3 m. One of the two machines is already in operation. The differential surge tank, Fig. 11, has a shaft diameter of 4·25 m., and an upper chamber 12·0 m. diameter. The penstock tapers from 2·0 m. to 1·7 m. diameter.

The Pavones scheme, some 5 km. downstream, comprises a dam 21 m. high, with a useful storage of 1·4 million cubic metres, receiving a largely controlled flow. The turbine will take 10 cu. m. per sec. under a head of 46·8 m., this being achieved by discharging along a canal 8·28 km. long.

Downstream of Pavones, the diversion dam will also receive a regular flow. The dam will be a spillway gravity structure, automatically maintaining the canal level. The canal follows the contour line of the northern boundary of the 12,000 hectares to be irrigated, to which some 125 million cubic metres of water will be supplied annually.

NOTE.—A description of Guadalén and notes on the author's tour appeared in the *Guilds Engineer* (of the Imperial College, London) for 1952.

Book Reviews

Civil Engineering Plant and Methods. By Rolt Hammond, A.C.G.I., A.M.I.C.E. Ernest Benn Limited, Bouverie House, Fleet Street, London, E.C.4. 1952. 229 pp. 41ff. 14 plates. Price 25s. net.

This book is in the nature of a causerie on current civil-engineering equipment and practice. It consists largely of sketch accounts of specific constructional schemes that are regarded by the author as holding some special interest, and of descriptions of plant and machinery, with due reference to manufacturers' specialities. This somewhat discursive treatment does not go deeply into the subject, nor is it comprehensive, but it provides a readable commentary on present-day trends in practical constructional work. The subjects considered include excavating plant, pilings and foundations, concrete mixing and placing equipment, cranes and lifting appliances, plant for dock and harbour construction, tunnelling methods and equipment, unit construction, welding, road-making machinery, and the organisation of civil-engineering work.

Annuaire de l'Association Suisse des Electriciens. (A.S.E.) Published by the "Association Suisse des Electricien," 301, Seefeldstrasse, Zurich 8. 126 pp.

This yearbook is a special issue of the well-known "Bulletin de l'Association Suisse des Electriciens," now published in common by the A.S.E. and "Union des Centrales Suisses d'Electricité" (UCS). The names and addresses of members are given in part I, and part II contains the byelaws of both Associations concerned, and a list of the A.S.E. publications.

Outdoor Switchgear. A leaflet from Cooke & Ferguson Limited illustrates outdoor switching units which were recently built for an Australian supply authority who required a two-switch 33 kV outdoor substation. As space was strictly limited, all protective equipment, control gear and auxiliary equipment had to be housed in the mechanism kiosks, and to ensure accessibility the mounting panels for instrument meters were hinged and the mounting of the remaining equipment arranged for easy withdrawal. A second leaflet gives details of the type OF.1 800 A 66 kV outdoor oil circuit breaker which has been submitted to short-circuit tests at the KEMA testing station, Holland. Certificates have been obtained for ratings of 750 to 1,000 MVA, covering the requirements of existing contracts, and additional tests were carried out for the rating of 1,500 MVA.

From page 209

Some monthly advances obtained from the Mignano heading where the six rounds scheduled per 24 hours were often surpassed and up to nine rounds were made per day, are very striking:—

July 1950	290 m.	(950 ft.)
September 1950 ...	314 m.	(1,030 ft.)
October 1950 ...	300 m.	(985 ft.)
November 1950 ...	342 m.	(1,125 ft.)

In the Venafro heading the advance during November was of 356 m. (1,170 ft.).

The rate of progress on the Volturno tunnel is believed to constitute a record for tunnels of this type. During 236 working days 4,636 m. (15,200 ft.) of tunnel were excavated with a rock volume of about 90,000 cu. m. (317,000 cu. ft.). The records, obtained through the organisation of the works by Eng. Silvio Arrigoni, are as follows:—

Record advance per month: 356 m. (1,170 ft.) during October 1950 with an average of 13·60 m. (44·6 ft.) advance per day.

Record advance per week: 90 m. (295 ft.) during the week October 8—15, with an average of 15 m. (49 ft. 3 in.) advance per day.

Record advance per day: 21 m. (69 ft.) on the 29th November 1950.

From July 1, 1950, for the tunnelling work as described, twenty type RH-655W Atlas rock drills with pneumatic pushers and 180 Sandvik Coromant drill steels were used. During the work no interruption was caused by breakdowns and the consumption of spare parts is understood to have been insignificant. The average length of hole drilled by each Sandvik Coromant hard-metal-tipped steel was about 450 m. (1,500 ft.).

From page 214

from all participants if it is to be carried out to schedule. The tests at Grudie Bridge were completed, once all plant and equipment were in position, in only about five hours by the active help of the owners, their consulting engineers, and the various contractors concerned.

Acknowledgments

The North of Scotland Hydro-Electric Board and The British Pitometer Co. Ltd. are thanked for their kind permission to allow the publication of various details contained in this article.

Vickers VR.180, preproduction model, equipped with VUCD.180 universal cable dozer, on test

Vickers Rolls-Royce Tractor in Production

An interesting ceremony took place recently at the offices of Jack Olding & Co. Ltd., Hatfield, when Mr. H. Hopkinson, Secretary for Overseas Trade, "christened" the first VR 180 tractor to be made on a production basis. This tractor, which is the result of five years' intensive development and test, is a joint product of Vickers-Armstrongs Limited and Rolls-Royce Limited, and is being distributed and serviced throughout the world by Jack Olding & Co. Ltd. It is stated to be the first heavy track-type tractor to be designed and built in Britain, and will form a valuable accession to Great Britain in the drive for the international market in heavy earth-moving equipment, as well as for a home market which in the past has involved a drain on the country's dollar resources.

A brief illustrated note on the tractor was published in our April 1952 issue, but further details are now available. The aim of the designers has been to produce a track-type heavy tractor that should be an advance on any other machine in the world, and in some respects the design is revolutionary. This applies particularly to the track suspension, which is a springless articulated system which allows the tracks to follow the contours of the ground, thus giving full traction under all conditions and allowing the VR 180 to travel at a higher speed than has previously been possible with any similar machine. The tractor frame is carried on the track suspension at three points so that it cannot be strained by the track movements; furthermore, the articulation is such that the minimum of movement is imparted to the driver's seat. Rubber cushioning rings built into the track wheels also contribute to smooth riding.

The power unit has been specially designed for the tractor, and is a six-cylinder supercharged Rolls-Royce oil engine with dry-sump lubrication, having a belt horsepower of 180. It drives the track through a single-dry-plate clutch and six-speed constant-mesh gearbox, the final drive being through a differential gear and servo-operated multi-plate steering clutches. The drawbar horsepower is 150. The maximum drawbar pull on first gear is 29,500 lb. and on top gear a useful maximum working speed is attained of 9·7 m.p.h.

Great attention has been paid to ease of maintenance. Sealed lubrication is used wherever possible, and unit construction has been adopted throughout so that any unit can be removed for servicing and a spare unit inserted in its place. The engine, when removed, stands flat on the ground on its wide-based sump.

To enable the most effective use to be made of the VR 180 tractor a series of "matched" ancillary units has been designed, comprising a control unit, a 12 cu. yard scraper, a dozer, a towing winch, and logging equipment.

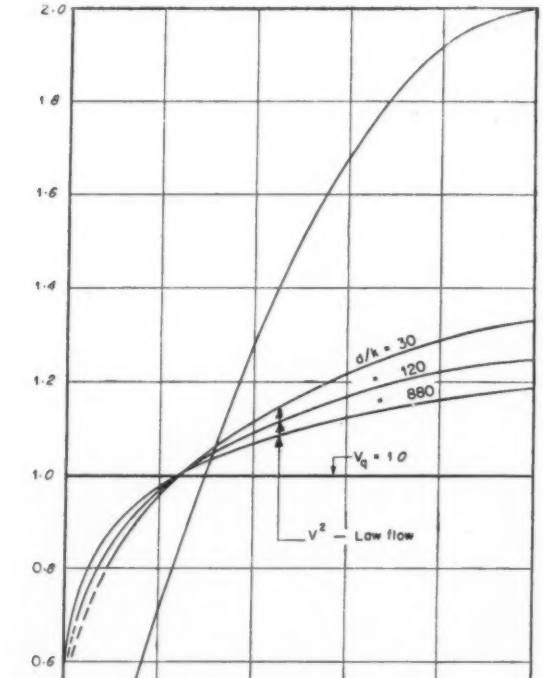
Preproduction models of the tractor and ancillary units have been exhaustively tested on many sites not only in Britain but overseas, under the most gruelling conditions that could be found. For instance, a tractor was tested in the Tripolitanian desert and others in Australia.

The immediate production capacity is 500 tractors per annum, and an increase to 1,000 per annum is contemplated. Orders are already being received from New Zealand, Africa and the Sudan, and Australia, in spite of the restrictions it has had to impose on imports, has sanctioned the purchase of 100 tractors during the next twelve months.

Abstracts from the World Technical Press

Laws of Flow in Rough Pipes

This article deals with a subject which is still being investigated by hydraulic research workers. The author does not proceed from head-loss measurements made on an experimental installation, giving the results obtained and comparing them with the various formulae in current use; he simply goes back to the source of various works published on this subject, and to the measurements generally used, particular attention being paid to the report on Nikuradse's tests published in Göttingen in 1933. From a close checking of the mass of experimental data given in the Nikuradse report, evidence is found which casts serious doubt on the accuracy of the Nikuradse formula of flow in rough pipes; further evidence against this formula is obtained from actual experience with large concrete pipes.



The author concludes that the Nikuradse formula should be discarded but that formulae of the Strickler and Manning types are fundamentally correct for small and large pipes in the V^2-law region of flow. He then analyses various types of flow and advocates a new "drag-velocity" formula for loss of head:—

$$S = 0{\cdot}0015 \frac{V_D^3}{V_0 R} \text{ m/sec.}$$

which is said to give results in agreement with those obtained by Manning; according to his new formula, a particular velocity occurs in the vicinity of the wall of the pipe, and the author claims that this velocity is in close relation with the appearance of turbulence in the flow. (James Williamson, M.I.C.E., *La Houille Blanche*, Vol. 6, No. 5, September-October, 1951, p. 738, 11 pp., 5 ff.)

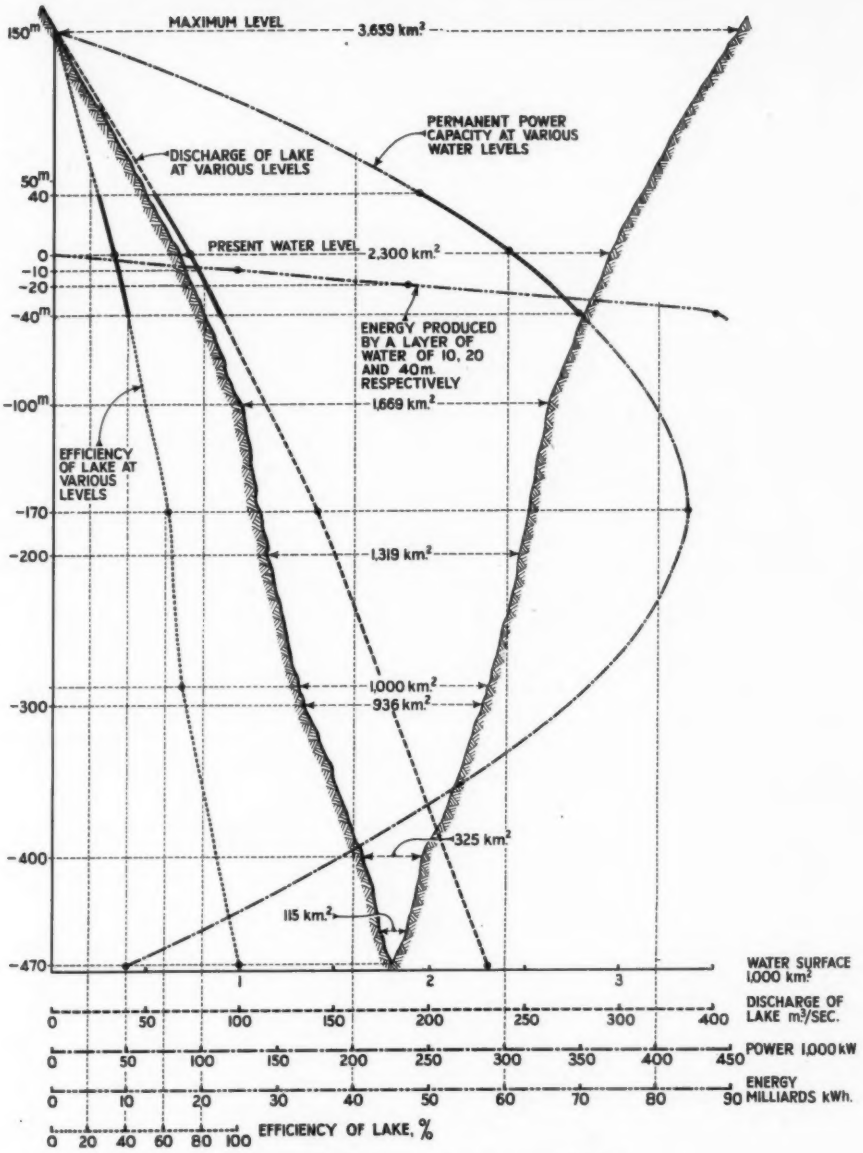
Harnessing Natural Lakes to Power Generation

This long survey is the synthesis of an article published in the *Revue Universelle des Mines* in October 1949, and of two papers read by the author in Brussels in February and May 1950. Its main basis appears to be investigations carried out at Lake Kiwu in the Ruanda-Urundi Territory, due west of the southern tip of Lake Victoria. A particularly interesting characteristic of this lake is that it was formerly a tributary of Lake Edward, which belongs to the Nile basin, and was cut off by the formation of the Virunga volcanic range; its waters then rose to their present level and subsequently found, through the Ruzizi valley, an outlet towards Lake Tanganyika, thus becoming part of the Congo basin.

The development of natural lakes for power production is possible only in instances where the two following conditions are fulfilled: (i) the lake must ensure that, even in protracted periods of drought, the discharge for which the power plant has been set up remains constant; (ii) it must be able to provide, at any time, an adequate storage capacity for excess water so as to avoid overflow losses. The first condition makes it possible to supply the total energy available in the lake at constant power, i.e. at the lowest cost per kWh. As to the second condition, we now know by experience that water has a value similar to that of fuel, and that hydro-electric plants should be designed and set up in such a way as to reduce water losses to a minimum. From this point of view plants arranged in series are not as advantageous as a single plant, owing to the overflow occurring at the various dams.

With an uninterrupted headrace, the turbines can utilise the total head in one drop, and there can be no overflow losses. If the lake supplying the power plant has an adequate storage capacity in reserve at any time, either, for example, throughout abnormally long rainy periods or in case of serious economical slumps, overflow losses are avoided; such a reserve capacity can be obtained without having to resort to the construction of a dam, that is to say without involving any expenditure, if it is possible to make use to a certain extent of the depth of the lake under water. Hence the advantage of using both the water collected by the lake and the water in storage.

As far as discharge is concerned, natural lakes act as regulators much in the same way as does the fly-



Characteristic curve for a natural lake

wheel of an engine which has to work off an irregular supply of steam, but whereas the flywheel absorbs energy the natural lake does not discharge the total volume of water it collects since a considerable part of it is lost by evaporation.

Evaporation losses increase, of course, in direct ratio with the area of the water sheet in contact with the air, and, besides, vary according to local conditions such as altitude, temperature, degree of dryness of the air, etc.

The author suggests a formula for ascertaining the mean discharge of a lake, the water surface of which can be varied between two extreme limits, either by raising the water level by means of a dam of adequate height so as to reduce the discharge to zero, or by lowering the water level by means of a bottom outlet so as to reduce the water surface to zero. In the case of Lake Kiwu the output is calculated as a function of the water level.

When, from the viewpoint of discharge regulation, the water surface of a lake is too large (this being shown by very slight variations of the water level), it may be expedient to reduce evaporation in order to improve the efficiency of the lake. To this end, the water level can be lowered systematically to give a smaller water surface and less evaporation. The water level may thus vary to a wider extent but these variations remain, however, within reasonable limits. After a lowering of the water level a higher mean discharge may follow; the available head is however reduced by the same amount.

By means of an example the author presents what

he calls the "hydro-electric paradox," showing that this procedure may lead to an apparent increase in power output, while for the type of lake instanced the power output would be reduced by raising the water level by means of a dam. In this particular case the advantages of a systematic lowering of the water level are fourfold:—

(1). Higher permanent power output, increasing, for example, from 300,000 to 419,400 kW at a 170 m. decrease in water level, it being theoretically assumed that the head loss of the plant is constant, and from 300,000 to 342,280 kW at a 70 m. decrease in water level at all discharges (therefore variable head loss). A stable working of the power plant is ensured between the two extreme levels given above since any additional lowering of the water level corresponds to an increase in power.

(2). Any decrease in the water level results in the water receding from the banks of the lake so that a wider area of land can be reclaimed for the benefit of the natives living on the shores of the lake. On the other hand, the setting up of a dam would unavoidably involve the flooding of wide tracts of land.

(3). Lowering the water level makes it possible to utilise the water stored in the lake to the greatest possible extent. In the case instanced the power capacity of the lake would rise 100 per cent., i.e., up to 600,000 kW, over a period of 40 years, and the additional capital expenditure required would be justified.

(4). Finally, the lowered water level makes available, at no cost whatsoever, a storage capacity equal in volume to the amount of stored water utilised. Excess water can thus be stored, and overflow losses are avoided.

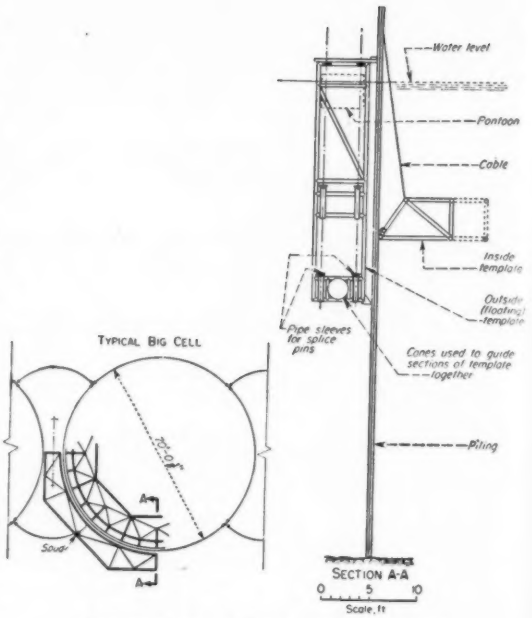
The main drawback of natural lakes is that in their undeveloped state they regulate imperfectly the outlet discharge which, at its minimum, is only a fraction of the mean discharge, e.g., 40 per cent. at Lake Kiwu. This problem has been solved at Owen Falls power plant by raising the water surface of Lake Victoria by means of a dam.

The author advocates the lowering of the water level, which dispenses with a dam, and avoids the loss of land on the shores. He states that if the level of certain lakes (Geneva, Erie) were further lowered by 1 metre, the minimum discharge would be equal to the mean discharge, and the permanent power output of these lakes could thus be doubled. (J. L. Créplet, *La Houille Blanche*, Vol. 6, No. 6, November-December 1951, p. 814, 29 pp., 21 ff.)

Driving 70-ft. Cofferdam Cells

Four cofferdam cells 70 ft. in diameter form the upstream cofferdam used by the Tennessee Valley authority for dewatering, for the purpose of power-plant expansion, a non-overflow length of the 36-year-old Hales Bar dam, below Chattanooga on the Tennessee river. These cells are second in size only to the two 76 ft. cells recently used in a similar operation in the Connecticut river at Holyoke, Massachusetts.

Added power at Hales Bar will come from added head. A crest length of the old dam has recently been raised so as to obtain water power for two new 24,300 kW units due to be in operation early this year; it is these units that were built behind the cofferdam in question.



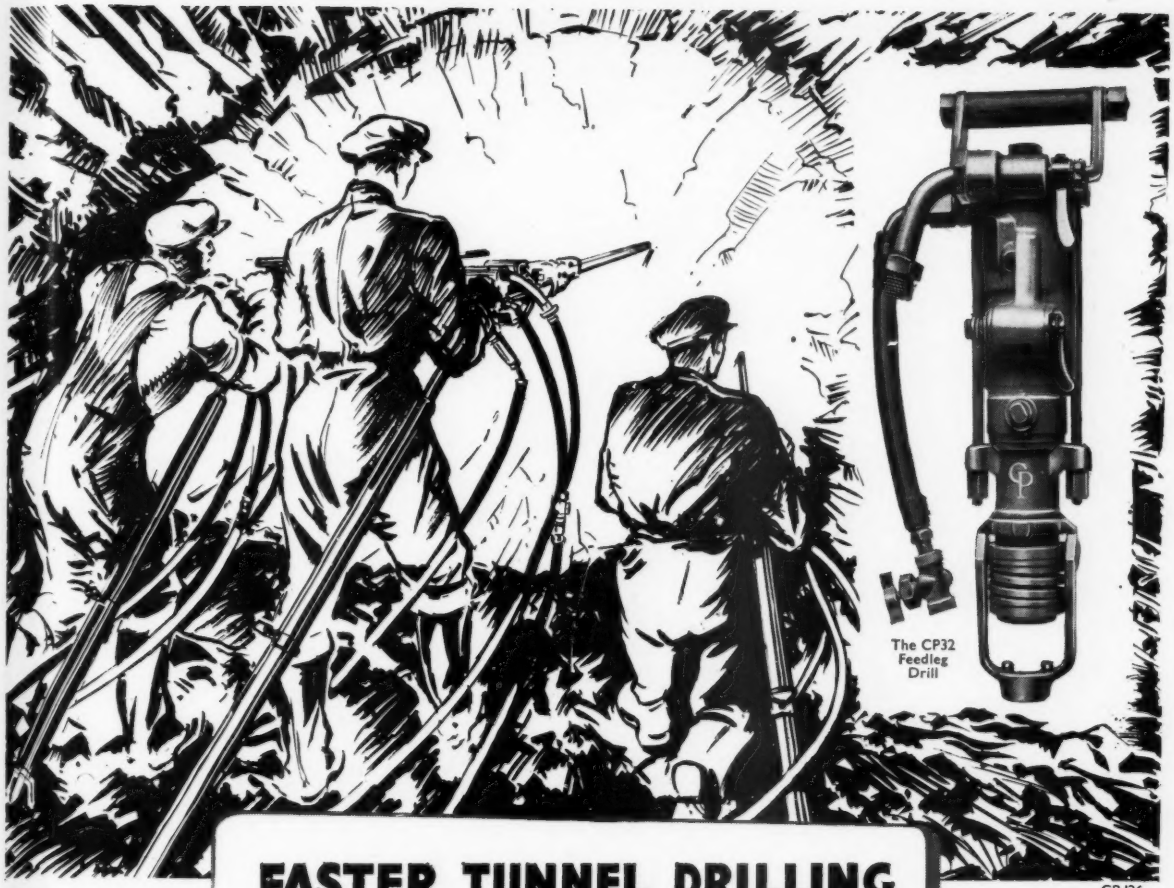
Construction of 70 ft. cofferdam cell

The key to success in accurately driving such large-diameter cells in 50 ft. of water was an elaborate exterior template, used in conjunction with a light special inside template. This structure of welded pipe fabrication gave a positive support to piling as it was placed in the cells. Putting the main template outside the sheets made it easy to handle, and more easily secured to the dam or to a completed cell. Pontoons built into the upper part of the template gave it the desired buoyancy, and a board floor on its top made an exceptionally good work platform all around the cell.

The cells were constructed as follows: The outer template was floated into place, assembled and anchored; four tee sections of piling were threaded into the guides and driven to rock. These tee sections kept piling vertical, and maintained proper spacing. The inner template was then suspended from tops of these tees. The remainder of the piling was then threaded into place, and driven with steam hammers handled by barge-mounted revolving cranes.

Before the templates were removed, the cells were partially filled with river gravel dredged a short distance below the dam and delivered to the cells by pipeline discharging through a rotary screen mounted above the cell, so as to remove some of the fines, and obtain a free-draining fill. This type of fill was desired to keep the saturation line down in the cells, for maximum shear resistance in the fill material, and minimum stress in the piling.

Once the space behind the cells had been dewatered and excavated to rock, a continuous concrete toe was dowelled to the rock and poured along the base of the cells as an added precaution against their kicking out at the bottom; seepage was carried to the sump through a channel formed in the top surface of the concrete. (O. R. Bengston, T.V.A., *Engineering News-Record*, Vol. 148, No. 3, Jan. 17, 1952, p. 34, 2 pp., 7 ff.)



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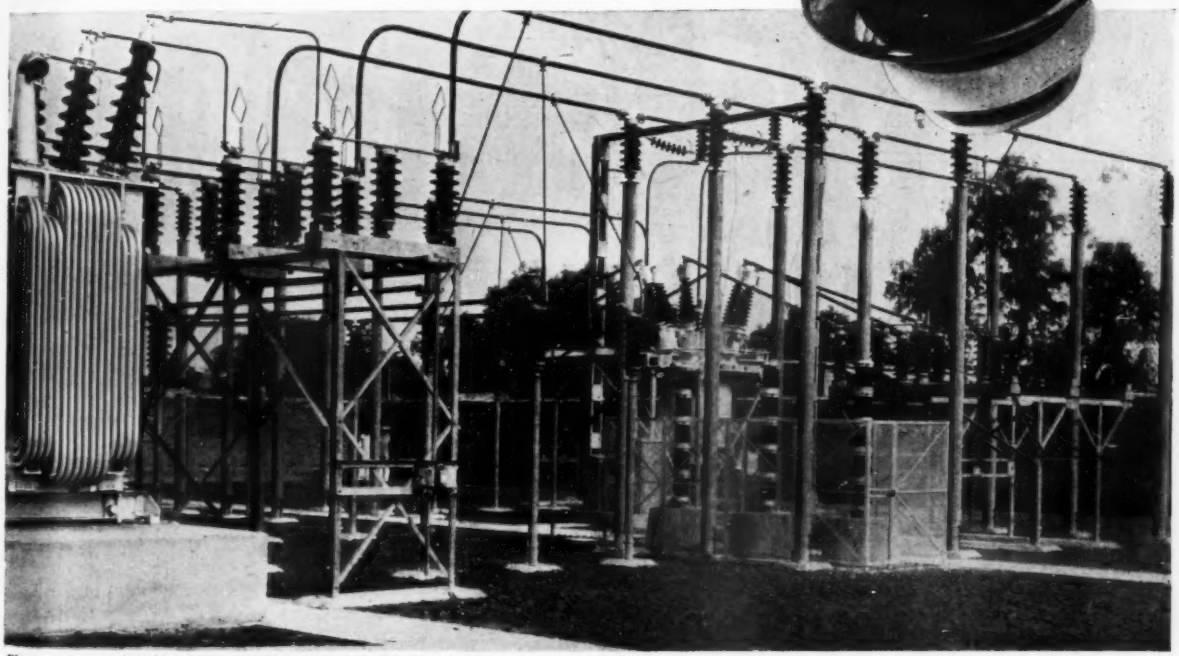

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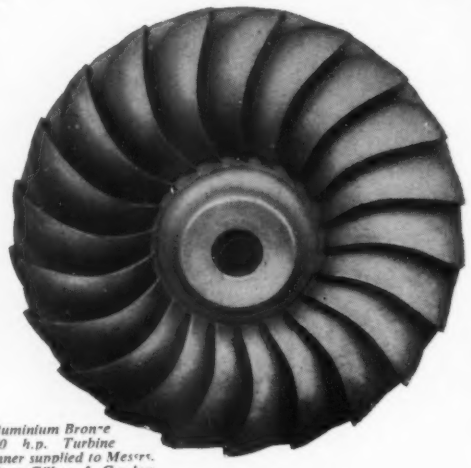

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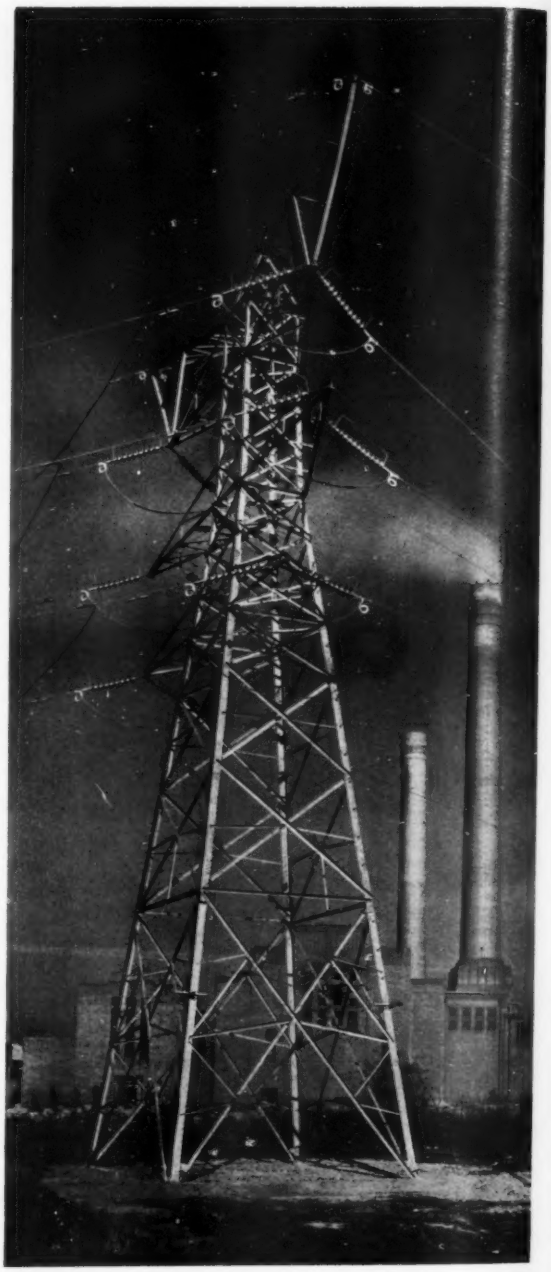

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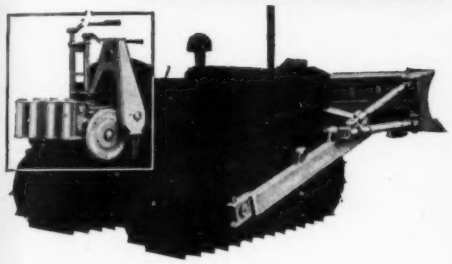

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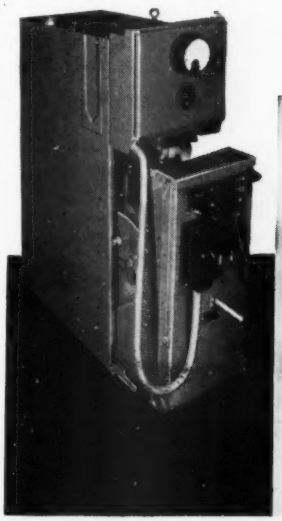

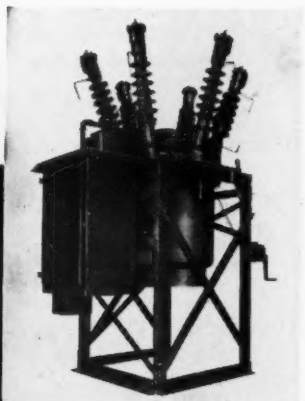

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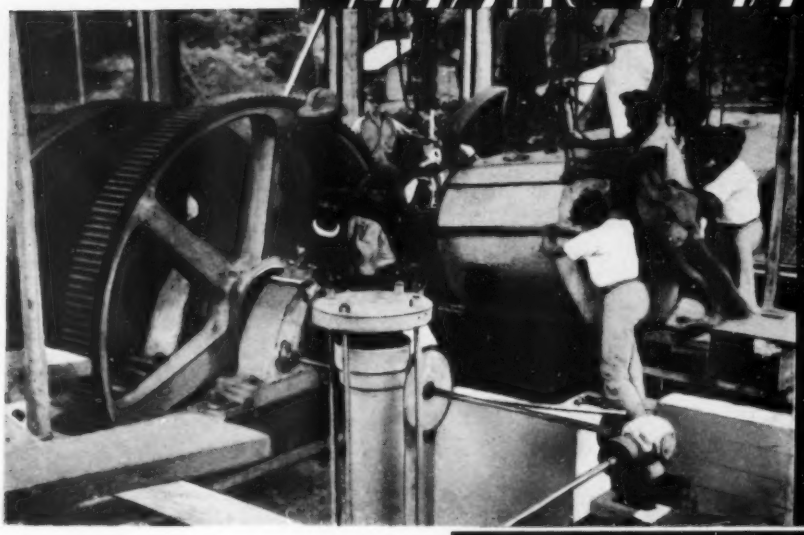

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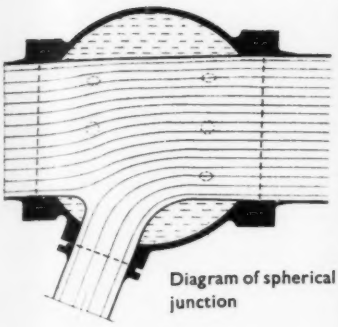
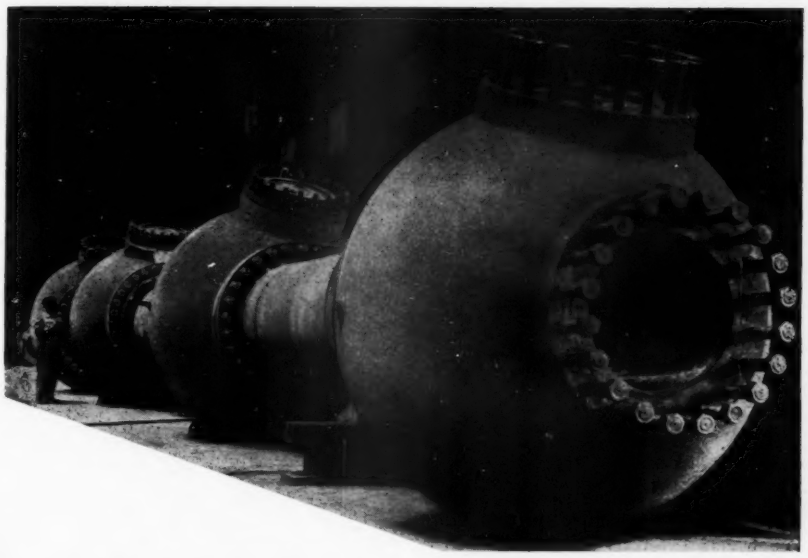


Diagram of spherical junction



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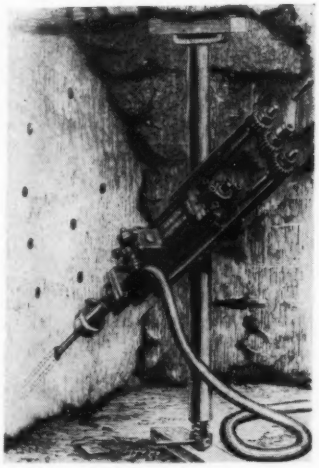

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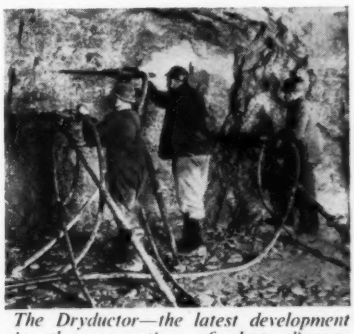

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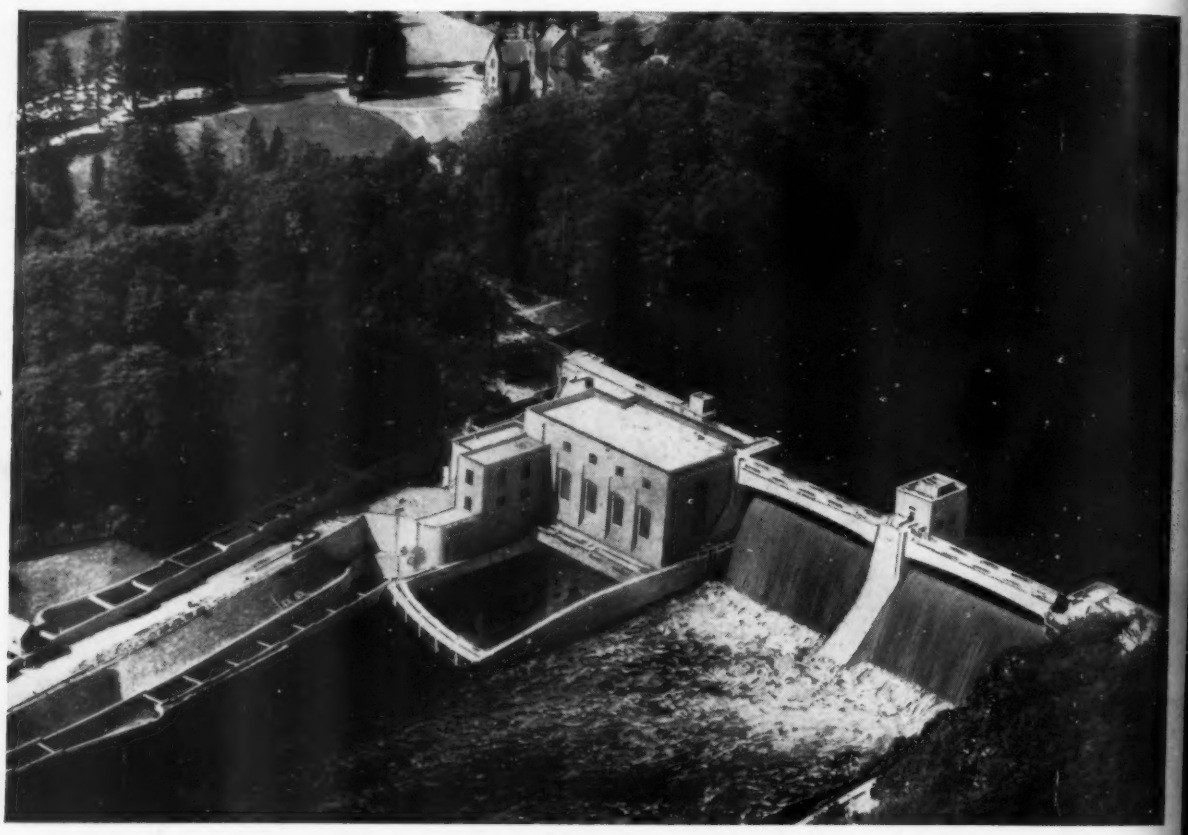

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